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NATIONAL DAM SAFETY PROGRAM. MEHERRIN RIVER DAM AT EMPORIA (VA --ETC(U)

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NSVILLE COUNTY, VA

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U.S. ARMY CORPS OF ENGINEERS
NORFOLK DISTRICT

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

(1)

MEHERRIN RIVER DAM
AT EMPORIA

TABLE OF CONTENTS

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

Name of Dam: Meherrin River Dam at Emporia

State: Virginia

River: Meherrin River

Date of Inspection: 19 April 1978

Based on the visual inspection, available records and stability calculations, the 43-foot high concrete Meherrin River Dam at Emporia is in a seriously deteriorated condition. The overflow spillway which is approximately 462 feet in length will not pass the one percent design flood without overtopping the right non-overflow section by approximately 2.5 feet. Therefore, spillway capacity is considered seriously inadequate.

Remedial treatment measures were implemented in 1977 in an effort to improve the stability of the dam. However, stability calculations for the spillway section indicate that the safety factor is inadequate for the 1 percent design flood.

In view of the concern for the safety of the Meherrin River Dam at Emporia, the following recommendations are presented for the Owner's consideration and implementation:

(1) Perform a detailed analysis of the downstream area to determine the impact limits of a possible dam failure. This analysis should be accomplished within 120 days after receipt of this report.

(2) Re-evaluate the stability of the dam for the 1 percent design flood, 1/2 PMF (Probable Maximum Flood) and PMF. Based on the results of this analysis, make recommendations to insure the stability of the dam under all conditions.

(3) Implement the recommendations derived from the stability analysis within 180 days if the flood impact study concludes that there is the potential for loss of life resulting from a dam failure.

(4) Develop a detailed emergency warning system to notify the downstream area of impending danger.

(5) Implement the sluice gates recommended by the Owner's consultant as soon as practical.

(6) Verify the effectiveness of the grouting program performed in 1977.

(7) Maintain a file of all available documents pertinent to the design, construction and operation of the Meherrin River Dam.

Until such time that the above recommendations can be implemented, the Owner should adopt the following policy:

(1) Provide round-the-clock surveillance of the Meherrin River Dam during periods of unusually heavy rains.

(2) When major storm warnings are given, the owner should activate his warning system procedures.

Approved:

Douglas L. Haller

DOUGLAS L. HALLER
Colonel, Corps of Engineers
District Engineer

MEHERRIN RIVER DAM AT EMPORIA - OVERVIEW

PLATE

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 August 1972 authorized the Secretary of the Army through the Corps of Engineers to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose of the Phase I inspection is to identify expeditiously those dams which might be a potential hazard to human life or property.

1.2 Project Description

1.2.1 Dam and Appurtenances: The Meherrin River Dam at Emporia, Virginia, is a 42.5-foot high (spillway crest elevation 112.4), 715-foot long, concrete gravity structure with a 462-foot long overflow spillway similar to an ogee type. The left end of the dam is an approximately 20-foot long non-overflow section with a top elevation of 120.2. The right end of the dam is a 137-foot long non-overflow section with a top elevation of 117.3. An abandoned power house is located between the right non-overflow section and the overflow spillway. The gate to the five power house intakes are inoperable with three in the closed position and two in the open position. The original turbines remain in place with their vanes locked in the closed position to keep the water from flowing through the power house. The power house is flooded with rain water. There are no regulating facilities for the dam. As a result, river flow is released directly over the spillway. Large amounts of siltation have been deposited upstream of the dam. The height of the siltation is reported to be within 11 feet from the top of the spillway. An old, inoperative fish ladder is located approximately in the center of the spillway.

1.2.2 Location: The Meherrin River Dam is located approximately one-half mile upstream of Interstate Route 95 along the boundary between the City of Emporia and Greenville County in Southeastern Virginia.

1.2.3 Size Classification: The dam is classified intermediate because of its storage and height.

1.2.4 Hazard Classification: The dam is located in an urban area and is, therefore, given a high hazard classification in accordance with guidelines contained in Section 2.1.2 of "Recommended Guidelines for Safety Inspection of Dams" published by the Office, Chief of Engineers. The hazard classification used to categorize dams is a function of location only and unrelated to the stability or probability of failure.

1.2.5 Ownership: City of Emporia.

1.2.6 Purpose of Dam: The primary purpose of the system is water supply for the City of Emporia.

1.2.7 Design and Construction History: The dam was constructed by the Greenville Electric Company in 1908 and acquired by the Virginia Public Service Company (predecessor to Virginia Electric and Power Company) at a later date. Sometime prior to 1940, the dam was raised from crest elevation 108 to elevation 112.5. At the same time, a parapet wall was added to the non-overflow section on the right bank raising it to elevation 117.3. In 1966, VEPCO stopped generating electricity at the dam and in 1970 sold the dam to the City of Emporia. In 1973, an engineering appraisal of the dam's stability was performed by Wiley and Wilson, Inc. (See Appendix V.) The study eventually lead to the installation of 30 rock bolts along the axis of the spillway to add stability to the dam. Several drill holes indicated large, vuggy voids in the dam and a grouting program was instituted through the anchor holes and an additional 30 grout holes.

1.2.8 Normal Operational Procedures: Other than debris removal, there are no operational procedures in effect.

1.3 Pertinent Data

1.3.1 Drainage Areas: The dam controls an approximate drainage area of 747 square miles.

1.3.2 Discharge at Dam Site:

Maximum known flood - 40,000 cfs in August 1940.

Ungated spillway, pool level at top of dam (Elev. 120.2) - 41,300 cfs.

Non-overflow section, pool level at top of dam (Elev. 120.2) - 3,400 cfs.

1.3.3 Dam and Reservoir Data:

Pertinent data on dam and reservoir is shown in the following table:

TABLE 1.1 - DAM AND RESERVOIR DATA

Item	Elevation ft, msl	RESERVOIR Area Acres	CAPACITY Acre-Feet 1/	CAPACITY Watershed Inches	Length Miles
Top of Dam	120.2	890	9,500	0.26	11.1
Non-overflow Section	117.3	730	6,900	0.20	10.2
Ungated Spillway Crest	112.4	510	3,800	0.12	8.7
Normal Riverbed	70.5	—	—	—	—

1/ Excludes an estimated 1,000 acre-feet of sediment.

SECTION 2 - ENGINEERING DATA

2.1 Design Data

2.1.1 Original Design: The only available document depicting the original design is a drawing of typical sections prepared by C. P. Burgwyn in 1908 for the Greenville-Water-Power Company. This drawing was traced and included as Drawing I in Appendix I. The typical sections indicated that reinforcing was placed in the upstream portion of the dam. The drawing also indicated that the original design designated a spillway elevation of 108 feet msl. It appears that anchors of some sort were specified on the upstream and downstream portions of the dam sections. However, the actual spacing is not specified. Explorations performed in 1977 by Wiley & Wilson, Inc. confirmed the existence of reinforcing; however, the existence of rock anchors was not verified. The existence of additional data pertaining to the original design is not known.

2.1.2 1946 Study: In 1946, VEPCO enlisted the services of Stone and Webster Engineering Corporation to develop plans and profiles for the dam and its appurtent structures. The results of their efforts are shown in Appendix I as Drawing II. A review of Drawing II indicated that the overflow section had been raised approximately 5 feet to elevation 112.4 prior to the 1946 study. Conversation with a past power plant operator indicate that this work was accomplished in 1913. The left and right non-overflow sections had been raised to elevations 120.2 and 117.3, respectively. Details of the power station are also shown on Drawing II. Additional data from the 1946 study are not available.

2.1.3 Stability: In 1973, the City of Emporia enlisted the services of Wiley and Wilson, Inc. to evaluate the stability of the dam and recommend remedial treatment measures. A detailed discussion of their stability analysis is included in Section 6: Dam Stability. The results of the stability analysis indicated that the dam would not be stable under a design flood equal to the Oct 1972 flood. As a result, the following recommendations were made: install rock anchors at 15' centers across the spillway; install two sluice gates in place of existing turbines to regulate flows and control silt levels; resurface the downstream face of the dam; and correct the erosion problem on the downstream slope of the left abutment. The installation of the rock anchors and related grouting is discussed in paragraph 2.2.

2.2 Construction Data

2.2.1 General: There are no known records of the original construction. There are also no records to indicate why or how the height additions were constructed. There are records available from 1950 to indicate that gunite repairs were proposed for the downstream face of the overflow section. An inspection by the Norfolk District

in 1963 verified that at least a portion of this work had been accomplished in an area, approximately 40 feet wide, adjacent to the power house. Except for this repair, no further additions, repairs or alterations were recorded prior to the work recommended by Wiley & Wilson in paragraph 2.1.3.

2.2.2 Rock Anchor Installation and Grouting: As a result of Wiley and Wilson, Inc. recommendations, a program to determine the suitability of the existing conditions for rock anchor installation was initiated in August 1976. The first phase of this program consisted of drilling seven core holes through the dam and into the foundation bedrock and testing random samples to determine their unconfined compressive strength. The holes were grouted at the completion of the drilling. The results of this drilling is attached in Appendix IV as Report No. 1. This report also discusses the general geology of the site. The seven holes were redrilled between 20 September and 1 October 1976. Anchors were placed in the holes, torqued and loaded to approximately 65 tons as indicated in Report No. 2 in Appendix IV. At the completion of the testing, the anchors were pressure grouted. All of the test anchors were satisfactorily grouted with the exception of anchor No. 7.

At the completion of the testing phase, twenty-three additional rock anchors were installed. The sequence was similar to that described above except that some of the bolts were grouted prior to torquing.

A majority of the anchor bolts were tested and grouted as designed. However, seven of the twenty-three were not properly grouted. Efforts to regrout these bolts proved futile, thus regrouting was abandoned. This final grouting attempt was completed on 4 March 1977.

As a result of the exploration and grouting performed during the installation of the rock bolt, an additional exploration and grouting program was recommended and initiated in February 1977. This additional program included two (2) holes in the north non-overflow area, three (3) holes in the south non-overflow area, and twenty-five (25) holes across the overflow section of the dam. The results of the additional grouting program is explained in detail in Appendix IV, Report No. 2. The total grout take for this additional grouting program was 596 cubic feet as compared to 949 cubic feet required during the installation of the rock bolts.

2.3 Operation Data

2.3.1 General: Past history indicates that the dam and its appurtenant structures were used to produce electricity from 1908 until 1966 after which time it was sold to the City of Emporia. The operational data during this period is not available. The City of Emporia currently withdraws raw water from the reservoir for water supply. Continuing siltation and debris buildup are continuous maintenance problems at the intake pipe and dam, respectively.

2.3.2 Design Floods: The dam has withstood two record floods: August 1940 and October 1972. The August 1940 flood crested at elevation 120.0 and is considered the flood of record. The October 1972 crested at elevation 117.2. No known structural deterioration was associated with either flood.

2.4 Evaluation

2.4.1 Design: Data from the original design is limited to a drawing showing typical sections. A geology report and stability analyses are not available to verify the validity of these sections. Design calculations related to the spillway raising to the present elevation are not available.

The evaluation of the stability analysis performed by Wiley and Wilson, Inc. is discussed in detail in Section 6 - Dam Stability. The only resulting recommendation that was implemented was the installation of rock bolts. The remaining recommendations, installing two gates for siltation control, resurfacing of the downstream face of the dam and correcting the erosion on the downstream slope of the left abutment, have not been implemented to date because of financing problems. However, implementation of these recommendations are considered essential to perserving the structural integrity of the dam.

2.4.2 Construction: No records are available to verify that the dam was constructed as indicated on Drawing 1 in Appendix I. Construction records for additional work performed prior to 1976 are also not available. This work included raising the spillway and guniting the downstream face of the dam.

The installation of rock bolts is evaluated in detail in Section 6 - Dam Stability. The success of the grouting program performed in 1976 and 1977 was never substantiated by either additional exploration or visual observations during periods of low flow. In addition, the program had limited penetration into the foundation bedrock. Considering the high degree of weathering encountered in the schist zones, an adequate foundation grouting program is considered imperative in minimizing seepage and uplift pressures.

2.4.3 Operation: A lack of operational data limits the evaluation of the dam's operational procedures. Because the existing equipment in the power house is in a state of total disrepair, there are no means of lowering the reservoir level or regulating the flows over the spillway. Consequently, the siltation behind the dam cannot be controlled. In addition, any proposed work on the downstream face of the dam will be hampered by flows over the spillway unless some means are devised for controlling these flows.

SECTION 3 - VISUAL INSPECTION

3.1 General: Prior to the field inspection performed on 19 April 1978, two previous inspections were performed in 1963 by the Norfolk District and in 1973 by Wiley & Wilson, Inc. Copies of their inspection reports are inclosed in Appendix V. In general, the inspections indicated that the dam was in poor condition with numerous points of leakage. Both reports highlighted the deterioration of the downstream face and crest of the dam.

3.2 Findings:

3.2.1 Dam and Abutments: The results of the 19 April 1978 inspection are recorded in Appendix III. A continuous flow over the spillway obscurred a majority of the downstream face of the dam. However, it was possible to inspect the downstream areas adjacent to both abutments. Inspection of the overflow sections in these areas confirmed the findings reported by the previous inspections. The concrete on the downstream face of the dam has deteriorated to the point where large irregular voids, approximately 12 inches in depth, highlight the face. The severe spalling and cracking is concentrated at what is believed to be the original concrete lift lines. The exposed aggregate within the spalled areas appears to be competent and free from excessive weathering. Observations of seepage during the inspections were obscurred by the flows over the spillway.

Serious erosion was noted on the downstream slope of the left abutment adjacent to the existing wing wall. It is believed that the erosion is the result of channeled surface runoff rather than flows over or through the dam. In addition to the erosion, numerous debris in the form of logs and limbs had accumulated in the downstream area adjacent to the left abutment and upstream of the dam. The debris on the upstream face of the dam was impeding the flows over the dam.

The visible rock, immediately downstream of the overflow section, has been smoothed by the flows but was basically sound. The exceptions were isolated zones of schist which in some cases were weathered to residual soils. A majority of the rock observed on both abutments was a diorite. The zones of schist were predominantly noted in the areas adjacent to the left abutment.

3.2.2 Power house: The concrete associated with the power house appeared to be in better condition than that noted on the downstream face of the overflow spillway. Flows through the five outlet channels under the power house are restricted by the original turbines which are locked in a closed position. In addition, three of the gates controlling the inflows to the penstocks are closed. The machinery associated with the operation of the gates and turbines is inoperable. Additional findings are described in Appendix III.

3.2.3 Reservoir Area: Observations of the reservoir area indicated that excessive silting has occurred adjacent to the immediate upstream shoreline. Conversations with Mr. McCord, Emporia City Manager, revealed that the dam has silted to within 11 feet of the crest.

3.2.4 Downstream Area: A U.S.G.S. quad sheet was used as a guide in an effort to visually inspect the areas which might possibly be affected by a dam failure. As a result of this visual inspection, it was concluded that the existing armory, the elementary school adjacent to the armory and a park area would suffer the greatest impact. In addition, several small businesses and approximately 10 houses might be affected by a dam failure.

3.3 Evaluation:

3.3.1 Dam and Abutments: It is not known whether the deterioration of the downstream face has affected the structural integrity of the dam. The spalled and cracked areas are certainly not a desirable feature and could possibly lead to further seepage paths. Past inspections indicate that numerous seeps have developed throughout the dam and its foundation. If allowed to continue these zones of seepage will affect the structural integrity of the dam. Likewise, if the erosion on the left abutment is allowed to continue unabated, the structural integrity will be placed in jeopardy.

3.3.2 Power House: Existing conditions prohibit the lowering of the reservoir level below the spillway crest. The low level outlets cannot be readily opened without extensive work and/or removal of the turbines. An operational gate would allow the lowering of the reservoir level and control of siltation behind the dam.

3.3.3 Downstream Area: A visual examination of the downstream area can only define the areas that might be affected by a flood wave resulting from a dam failure. A detailed downstream analysis will be needed to determine a more accurate impact area.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures and Maintenance: There are little to no operating procedures for the dam. Large amounts of water-borne debris are caught by the dam and removed irregularly. The abandoned power house is locked and flooded. The building lacks maintenance. Operating and maintenance manuals and records are not kept.

4.2 Warning System: There is no warning system maintained by the City of Emporia.

4.3 Evaluation: The dam does not require an elaborate operational and maintenance program. Maintenance or resurfacing of the dam's downstream face is discussed in other sections.

SECTION 5: HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: There is no original hydraulic or hydrologic design data available for the Emporia Dam.

5.2 Hydrologic Records: Flow records on the Meherrin River have been maintained at Emporia (drainage area 747 square miles) by the Virginia State Water Control Board (S.W.C.B.) since January 1951. This gaging station is located 0.8 mile downstream of the dam. The U. S. Geologic Survey (U.S.G.S.) has maintained flow records on the Meherrin River near Lawrenceville (drainage area 552 square miles) since 1928. The U.S.G.S. gaging station is located approximately 19 miles upstream of the dam. Locations of these gages are shown on watershed map in Appendix II. Flow records at the dam were maintained by VEPCO during its ownership of the dam.

5.3 Flood Experience: A list of floods exceeding 15,000 cfs at either of the above described gaging stations is shown in the following table:

TABLE 5.1 MEHERRIN RIVER FLOODS

Date of Peak at Lawrenceville	Lawrenceville Drainage Area 552 Sq. Mi.	Emporia Drainage Area 747 Sq. Mi.
2 Jun 1889	18,000	—
28 Aug 1908	19,000	—
27 Apr 1937	17,300	--
17 Aug 1940	38,000	40,000
26 Oct 1971	17,700	19,400
7 Oct 1972	20,000	21,100
16 Jul 1975	11,800	16,200

The flood of August 1940 is known to have been the largest flood since at least 1873 at both Emporia and Lawrenceville. A discharge of 38,000 cfs was recorded near Lawrenceville while a flow of 40,000 cfs was estimated from highwater marks at Emporia. VEPCO recorded a reservoir pool elevation of 120.0 and a tailwater elevation of 104.5 for 1940 flood. This flood is estimated to have been a one percent flood at Emporia.

5.4 Reservoir Regulation: A spillway rating was computed for the 462-foot long, sharp crested spillway and a reservoir storage capacity curve was developed from U.S.G.S. Quadrangle Maps. Backwater calculations were performed from downstream of the S.W.C.B. gage to the face of the dam to obtain the tailwater elevation.

The average flow of the Meherrin River at the dam site is 664 cfs with a median flow of approximately 250 cfs. Mediam flow conditions produce a reservoir pool level slightly above the spillway crest with flow over the spillway most of the time. There is no flood control storage space designed into the reservoir.

5.5 Flood Potential: A Flood Plain Information (F.P.I.) report and a Flood Insurance Study (F.I.S.) have been completed by the Norfolk District, Corps of Engineers, for the City of Emporia in 1964 and 1976, respectively. These reports detail the flood hazard along the Meherrin River in Emporia.

A unit hydrograph was developed from the October 1971, October 1972 and July 1975 storms and was used to reconstitute the 1940 flood. Synthetic rainfall was applied to the unit hydrograph to assess the flood potential at the dam. The large drainage area upstream of the dam and the limited amount of surcharge storage available in the reservoir precludes any significant reduction in flows by the reservoir, particularly for the larger flows.

The rainfall applied to the developed unit hydrograph was obtained from the National Weather Service publications Hydrometeorological Report No. 33 and Technical Paper No. 40 for the Probable Maximum Flood (P.M.F.) and one percent flood, respectively.

5.6 Overtopping Potential: The probable rise in the reservoir and other pertinent information is summarized in the following table:

TABLE 5.2 RESERVOIR PERFORMANCE

	Median Flow	FLOOD One Percent 1/	1940	1/2 PMF	PMF 2/
Peak Flow, cfs	250	39,500	40,000	52,000	104,000
Peak Elevation, ft, msl	112.8	119.9	120.0	121.2	125.7
Ungated Spillway					
Depth of Flow, ft	0.4	7.5	7.6	8.8	13.3
Avg. Velocity, fps	1.4	10.7	10.7	11.7	14.5
Non-overflow Section (elev. 117.3)					
Depth of Flow, ft	0	2.6	2.7	3.9	8.4
Avg. Velocity, fps	0	6.1	6.1	7.8	11.5
Non-overflow Section (elev. 120.2)					
Depth of Flow, ft	0	0	0	1.0	5.5
Avg. Velocity, fps	0	0	0	3.5	9.1
Tailwater Elevation, ft msl	72.4	104.6	104.6	108.4	118.4

1/ The One Percent Exceedence Frequency Flood has one chance in 100 of being exceeded in any given year.

2/ The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

5.7 Reservoir Emptying Potential: There are no existing methods available for drawing down the reservoir pool level below the spillway crest.

5.8 Evaluation: The non-overflow section of the dam was overtopped by 2.7 feet in the August 1940 flood. It would be overtopped by about 3.9 feet in the 1/2 PMF flood and 8.4 feet in the PMF flood. The relatively short section designated the top of the dam would be overtopped by 1.0 and 5.5 feet in the 1/2 PMF and PMF flood, respectively. The flood hydrographs utilized in the above analysis are based on runoff characteristics demonstrated in actual floods and are deemed to represent the 100-year and PMF floods fairly accurately.

SECTION 6 - DAM STABILITY

6.1 Stability Analysis: The original stability analysis cannot be found. In 1974, after a site inspection and taking what measurements could be made without dewatering 1/, Wiley & Wilson performed a stability analysis on the spillway section. This analysis is on file at the Norfolk District, Corps of Engineers. Based on the results of this analysis, it was concluded that the dam was only stable under normal flow conditions and the then present level of siltation (11 feet from top of spillway). The vertical and horizontal dimensions adopted for the spillway section were 38.5 and 23.5 feet, respectively. This analysis did not consider uplift pressures. Geotechnic, Inc. was contracted to evaluate the integrity of the dam and the suitability of the foundation for rock bolt placement. Tensioned rock bolts were designed to increase the stability of the dam using a maximum flood level of 5 feet 6 inches above an assumed spillway crest elevation of 111.8. This design was accomplished without uplift considerations. Two-inch diameter Williams rock bolts were recommended and installed at 15 feet on centers across the spillway portion of the dam and tensioned to 65± tons. Geotechnics, Inc. report indicates the rock bolts were tested at the 65 ton loading. Grouting of the bolts encountered much difficulty with 8 bolts receiving less than 2 cf of grout and others receiving various amounts up to 4.5 cf. Even though the boring logs reveal a severely weathered contact plane at the base of the dam, the sliding resistance of the dam was not studied. The remedial design uses a safety factor of 1.25 against overturning. The silt level was assumed to be maintained at 11 feet below a spillway crest elevation of 111.8.

6.2 Foundation Conditions:

The foundation of the dam across the river valley and both abutments is relatively sound bedrock. Based on boring logs prepared by Geotechnic Inc. in connection with the latest remedial work and on the Corps of Engineers visual inspection, the bedrock consists of predominately hard, competent hornblend diorite with zones of less competent chlorite schist. Other rock types including diabase, hornfels and chloritic diorite also occur in minor amounts.

The dam site is located in the Piedmont Igneous-Metomorphic Belt of Virginia. Very little geologic mapping has been done in the Emporia area, however, it is believed that the rocks are part of the Petersburg granite intrusion of late precambrian age. This intrusion is composed perdominately of granitic rocks, however, other igneous intrusives such as diorite are know to occur locally. The existance

1/ From a telephone conversation between J. Irving, Corps of Engineers, and C. Dodal, Wiley & Wilson.

of minor amounts of metamorphic rocks within the intrusion are common as some contact metamorphism took place within the intrusive rocks and between the intrusive rocks and intruded country rock. At the dam site this is evident as chlorite schist, a metamorphic rock, is found occurring in the predominate diorite igneous intrusive rock.

The diorite comprises most of the right abutment and valley bottom with the schist zones occurring in steeply dipping zones of varing thicknesses. The left abutment is comprised of what appears to be predominately chlorite schist with occational interlayered zones of diorite. However, this information is based on the borings done during the remedial work and not on geologic mapping before construction. It is possible that the borings drilled on the left abutment were drilled entirely through steeply dipping zones of chlorite schist occurring in the predominate diorite bedrock. This interpretation would more correspond with the geologic conditions evident under most of the dam. A second area where chlorite schist appeared to be dominant was approximately 30 feet north of the fish ladder under the river valley.

The condition of the bedrock foundation varies from competent, unweathered diorite to highly weathered chlorite schist. The diorite is a harder, more weathered resistant rock as evidenced by the borings and several unconfined compression tests run by Geotechnics Inc. Some fracturing was noted in the borings but highly broken, weathered zones were not encountered except in isolated areas at the dam - foundation contact. Pronounced jointing within the diorite was not indicated on the drill logs or observed during the visual inspection. The chlorite schist which appeared to occur in zones a few inches thick to several feet, is a softer, less weather resistant rock than diorite. This was also confirmed by the borings and unconfined compression tests. Foliation or schistosity of the chlorite schist is well pronounced in the rock cores and outcrops downstream of the dam. The strike of the schistosity as well as the strike of the cones themselves appear to be approximately perpendicular to the dam alignment and dip steeply toward the left abutment. The schist varies in hardness depending on the degree of weathering. Most of the schist in contact with the dam is highly fractured and badly weathered and generally becomes less fractured and weathered with depth. A notable exception is approximately 30 feet north of the fish ladder where boring No. 5A was drilled. The schist in this boring was highly weathered to the point of being friable for a thickness of 12 feet below the dam contact. Numerous vugs were also pronounced along the foliation planes denoting solution weathering. The chlorite schist is composed of platy minerals which are aligned along foliation planes and which usually have low cohesion values and angles of internal friction. Schist zones where interbedded with stronger rocks like diorite form zones of weakness and may be highly susceptible to sliding if the orientations

of the zones dip at low angles either upstream or downstream. Because the schist zones at the dam site strike approximately perpendicular to the dam and dip steeply toward the left abutment, potential sliding along these zones is not anticipated and therefore not considered critical to the dam stability.

Seepage through the chlorite schist zones in the foundation rock does appear to be a potential problem. The orientation of the schist zones, striking perpendicular to the dam, provides a possible seepage path under the dam. This orientation as well as the low weather resistant nature of the schist are conditions which may readily contribute to foundation seepage. In conclusion, the foundation conditions appear to be relatively good with the exception of the seepage potential along locally, highly weathered zones at dam-foundation contact and through the upstream-downstream striking schist zones.

6.3 Evaluation

6.3.1 The stability analysis for this dam cannot be completely evaluated under the Office of the Chief of Engineers "Recommended Guidelines for Safety Inspection of Dams," National Program of Inspection of Dams (Vol. 1, App. D). The guidelines do not discuss rock anchors; therefore, Anchoring in Rock by Hobst and Zajic, published by Elsevier Scientific Publishing Company in 1977 was used as a reference for rock anchor design and installation.

6.3.2 The stability analysis performed by the owner's consultant and the ensuing remedial measures have many aspects which neither conform to the Corps of Engineers criteria nor the state-of-the-art in dam design. These aspects are discussed in detail in the following paragraphs.

6.3.2.1 The vertical and horizontal cross sectional dimensions used are not consistent with the core borings. The analysis uses a bottom elevation of 73.3. Many of the boring logs establishes the concrete-rock contact between elevations 69 to 71. In this type of remedial analysis, the most conservative figures should be used.

6.3.2.2 Uplift considerations have been ignored throughout the analysis. Design criteria differ in the amount of uplift to be considered in a stability analysis, but all criteria agree that uplift must be included. The Corps guidelines require total uplift be used in a stability analysis. Visual observations of the dam (seeps, severely weathered exterior, etc.) and core borings indicate that the dam and foundation rock contact is severely weathered in several large areas. Therefore, the condition of this dam requires full uplift be used in the stability analysis.

6.3.2.3 The maximum flood condition considered assumes a headwater 5.5 feet above spillway elevation of 111.8. Tailwater is neglected. This condition is less than a 100 year event and, also, less than the largest known flood of record, 1940, which had a recorded headwater 7.6 feet above the spillway. Under Corps criteria, the dam is classified intermediate size, high hazard and should be analyzed for the probable maximum flood (PMF).

6.3.2.4 The Williams rock bolts were tensioned and tested at 65± tons. The design uses the 65 tons as the working load. Rock bolt theory requires three levels of stress to be considered 1/: the basic prestressing force which is required to verify the anchor's strength and the rocks ability to hold the anchor, the initial anchoring force which exists in the anchor after the tensioning equipment has been disengaged and the ultimate prestressing force which is used in the static analysis. Because the anchors were tested at 65± tons, this must be considered the basic prestressing force. The ultimate prestressing force is considered to be approximately 65 percent of the basic prestressing force. The stability analysis should have used a maximum tension force of 42 tons. It has not been demonstrated the rock foundation can resist higher loads. To try and use a higher load would reduce the safety factor. Additionally, Williams Form Engineering Corporation recommends their 2" Ø bolts be tensioned to 74 tons and a design load under average conditions to be 2/3 of this load or 49 tons. The stability analysis has used too large a value for the rock bolt force.

6.3.2.5 Four rock anchors 2/ received less than 1/4 CF of grout after tensioning. Of primary importance in rock anchor installation is corrosion protection. Without this protection the useful life of the bolt is severely shortened. The grout protection also helps the anchor bond to the rock and concrete. The effectiveness of these anchors cannot be assured and should not be used in the analysis.

6.3.2.6 A sliding analysis was not performed for the dam. The core borings describe the contact plane between the dam and rock as highly weathered, fractured and vuggy with many borings having low (less than 50%) core recoveries. All of this would indicate the need for a detailed geology report with particular emphasis on joint patterns and weathered zones. Friction and cohesion coefficients should be established and used to determine sliding stability.

1/ Page 165, Anchoring in Rock

2/ Geotechnics, Inc. Report of Core Drilling, Grouting, and Rock Anchor Installation, 1977. (Appendix IV, Report 2)

6.3.2.7 A safety factor of 1.25 was used for overturning stability. Based on the in situ condition of the dam and rock foundation, a larger safety factor should be provided. The reference noted in paragraph 6.1 recommends a safety factor of 1.5.

6.3.2.8 The analysis assumes the silt will remain at its present level. Even though the owner's consultant recommended the installation of gates which could be used to flush the silt periodically, this plan has not been implemented. For purposes of stability analysis, siltation should be assumed to increase until measures are instituted to control silt levels.

6.3.3 As has been dicussed, the stability analysis has failed to recognize several important factors used in the design and analysis of dams. The consultant has not issued a report which summarizes the effect the remedial measures have had on the dam. It cannot be concluded from the stability analysis that the dam is stable either for overturning or sliding under a proposed 100 year or 1 percent flood.

SECTION 7 - DAM ASSESSMENT

7.1 Safety: A lack of design and construction data limited the evaluation of the dam to operation history and recent engineering studies. Existing records indicate that the dam withstood the 1940 flood which is the flood of record and is approximately equal to the 1 percent design flood. However, the concrete in the dam and portions of the underlying bedrock have undergone extensive deterioration as evidenced by recent investigations. The Owner's consultant performed a stability analysis and concluded that the dam would not be stable under a design storm equivalent to the October 1972 storm. As a result of this conclusion, the consultant recommended several remedial measures of which only the rock bolt plan was implemented. However, a review of this analysis, detailed in Section 6, indicates that the stability of the dam for the above design storm has not been assured by the implementation of the rock bolts. It should be further noted that the additional remedial measures which included the installation of two sluice gates to control siltation and reservoir levels were not implemented.

7.2.1 Flood Impact Study: It is recommended that the Owner enlist the services of a qualified consultant to analyze the downstream area and to define the area affected by a flood wave resulting from a dam failure. The analysis should determine the effects of a failure at the following pool levels: normal, 1 percent storm, 1/2 PMF and PMF. Emphasis should be placed on the estimated property damage and potential loss of life. In addition, the effect of a failure on the existing water supply should be addressed. The recommended analysis should be completed within 120 days after receipt of this report.

7.2.2 Stability Analysis: It is recommended that the Owner enlist the services of a qualified consultant to re-evalate the stability of the dam for the following design conditions: normal pool, 1 percent storm (1940), 1/2 PMF and PMF. This re-evaluation should be performed concurrent with the flood impact study. The design loads and assumptions detailed in Section 6 should be addressed in the analysis. Also, the effects of the rock bolts installed in 1976 and 1977 should be reflected in the analysis. Based on the results of the analysis, the consultant should make recommendations to insure the stability of the dam under all conditions. The problem of erosion on the abutments should be addressed with recommended solutions. If the flood impact study recommended in paragraph 7.2.1 concludes that a dam failure will possibly result in loss of life, then the recommendations resulting from the re-evaluation of the dam's stability should be implemented within 180 days after the published date of this report.

7.2.3 Warning System: An emergency warning system should be developed as soon as possible to notify the downstream inhabitants of an impending dam failure.

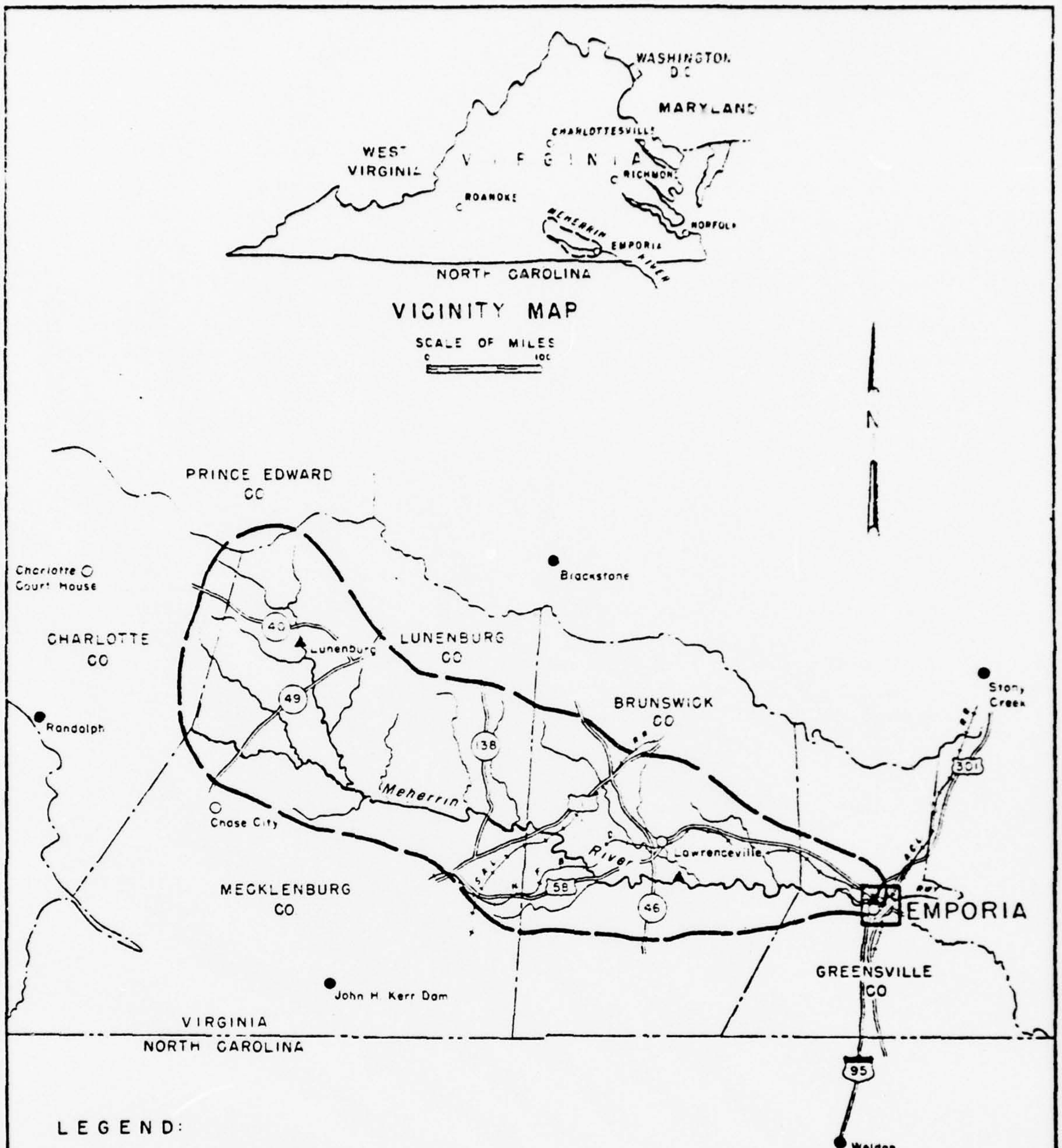
7.2.4 Remedial Treatment: It is recommended that the sluice gates recommended by Wiley & Wilson be installed as soon as possible to provide a positive means of lowering the reservoir level and controlling the level of siltation. Erosion adjacent to left abutment should be corrected as soon as practical.

7.2.5 Grouting: The effectiveness of the grouting performed in 1976 and 1977 should be evaluated during periods of low flow and/or by additional exploration and grouting.

7.2.6 Design Documents: A complete set of available design documents should be maintained by the Owner. These files should include available design drawings, calculations, pertinent correspondence and maintenance records. It is further suggested that the Owner implement a periodic inspection program to determine if the noted deterioration and seepage is progressing.

APPENDIX I

MAPS AND DRAWINGS



LEGEND:

General limits of flood plain study area as shown on Plate 2

Limits of basin

Existing hydrologic stations:

Recording	*Non-recording*	
▲	△	*Stream gage*
●	○	*Precipitation*

FLOOD PLAIN INFORMATION REPORT
MEHERRIN RIVER AT EMPORIA, VA

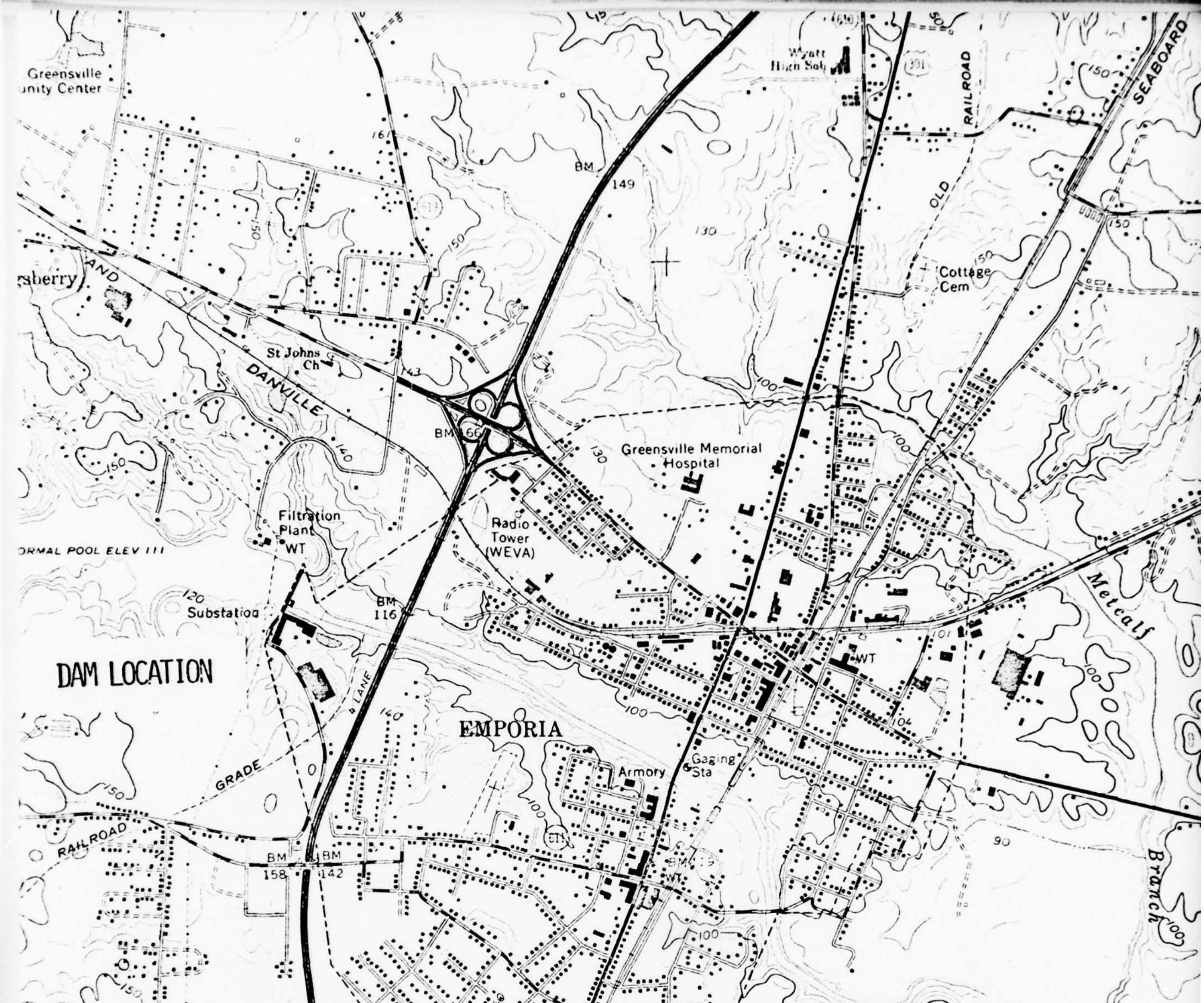
BASIN LOCATION MAP

U.S. ARMY ENGINEER DISTRICT, NORFOLK

DRAWN BY M.R.H.
CHECKED BY J.R.P.

FEB 1964
C-64-10-01(1)

PLATE 1



VICINITY MAP
MEHERRIN RIVER DAM
AT EMPORIA

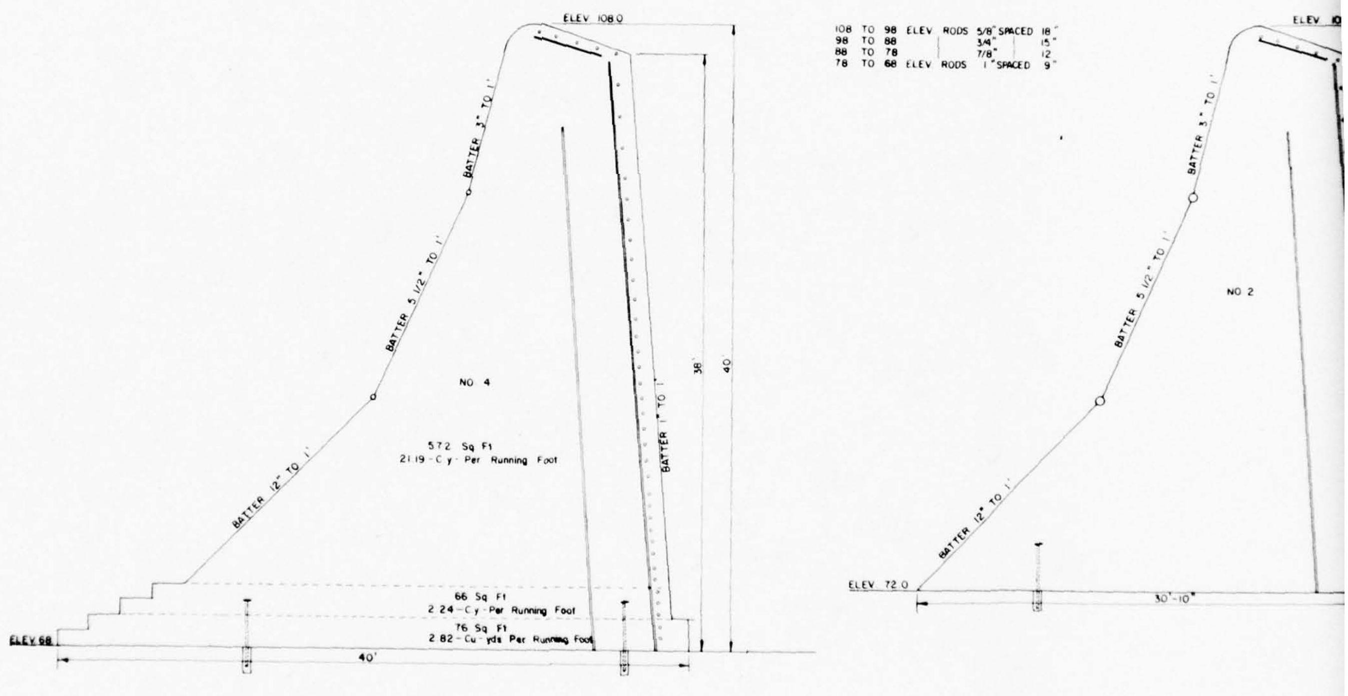
TRACED BY R.E.K. MAY 1978

CROSS SECTION - SPILLWAY OF DAM
GREENSVILLE - WATER - POWER CO.

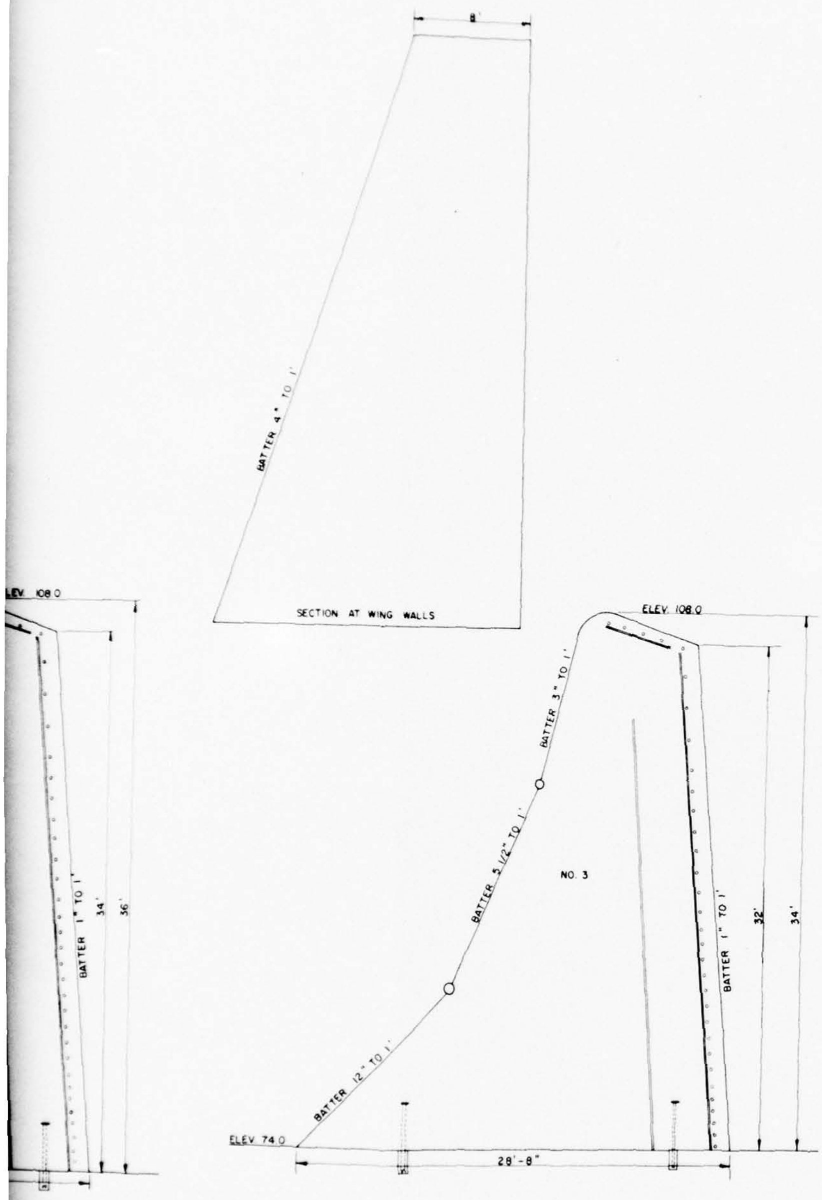
DESIGNED BY C.P.E. BURGWYN C.E.

SCALE 1" = 4'-0" OCT 1908

NOTE
NO 1 SECTION WHEN ELEVATION OF BOTTOM IS 68
NO 2 72
NO 3 SECTION WHEN ELEVATION OF BOTTOM IS 74



U. S. ARMY

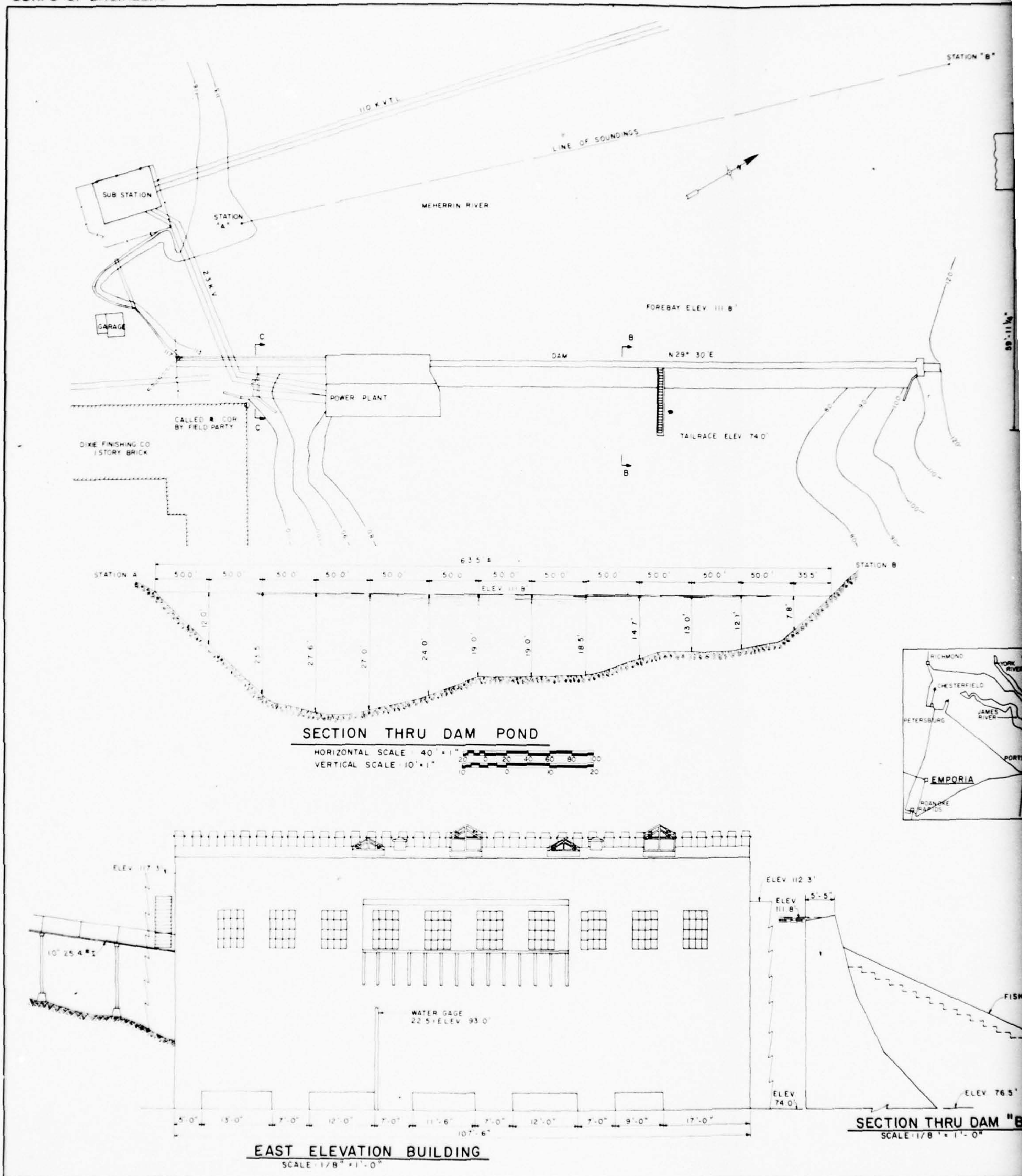


MEHERRIN RIVER DAM
AT EMPORIA

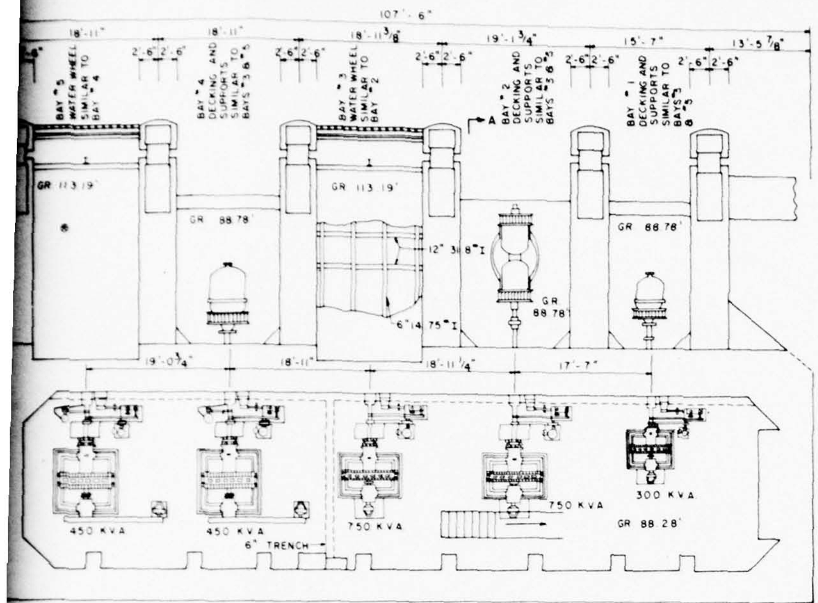
2

NORFOLK DISTRICT FILE NO.

CORPS OF ENGINEERS

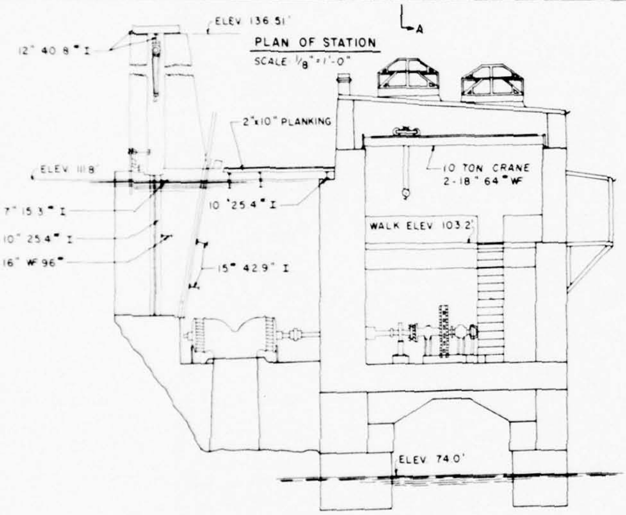


U.S. ARMY



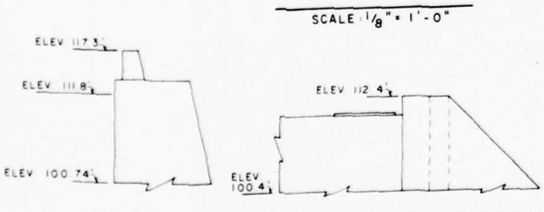
PLAN OF STATION

SCALE 1/8" = 1'-0"



SECTION "A A"

SCALE 1/8" = 1'-0"



SECTION "CC" THRU DAM AND HEADWALL

SCALE 1/8" = 1'-0"

GENERAL PLAN & SECTIONS — EXISTING STRUCTURE

HYDRAULIC STRUCTURE

EMPORIA HYDRO-ELECTRIC STATION - EMPORIA, VIRGINIA

VIRGINIA ELECTRIC & POWER CO.

STONE & WEBSTER ENGINEERING CORPORATION

SCALE GIVEN
JULY 12, 1946
ACCT 7 VEPCO

CHKD	INSP	COR	APPR

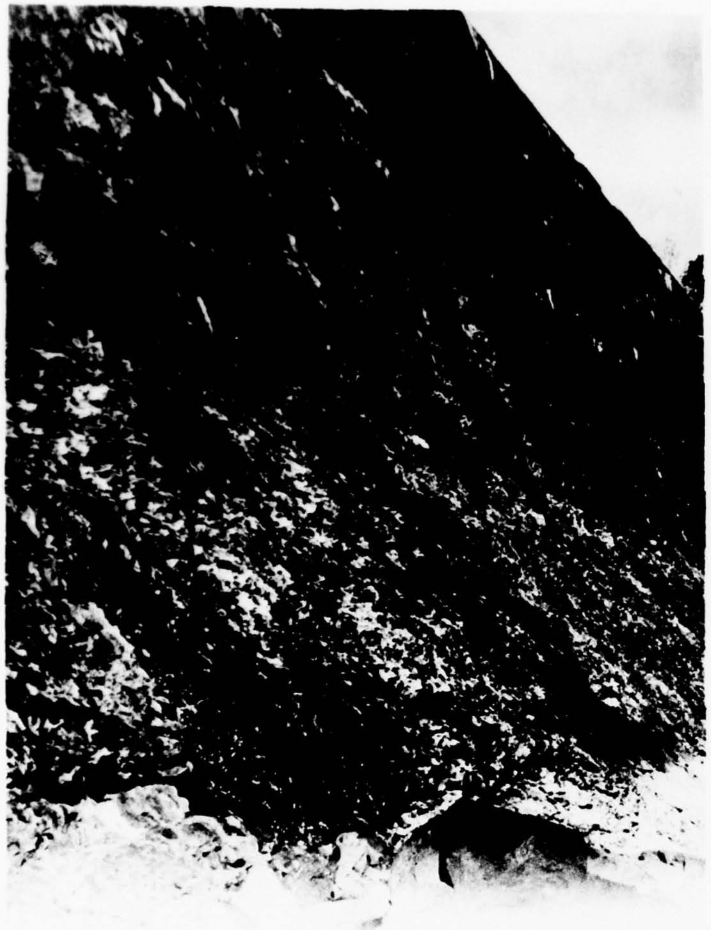
F SH NO 28

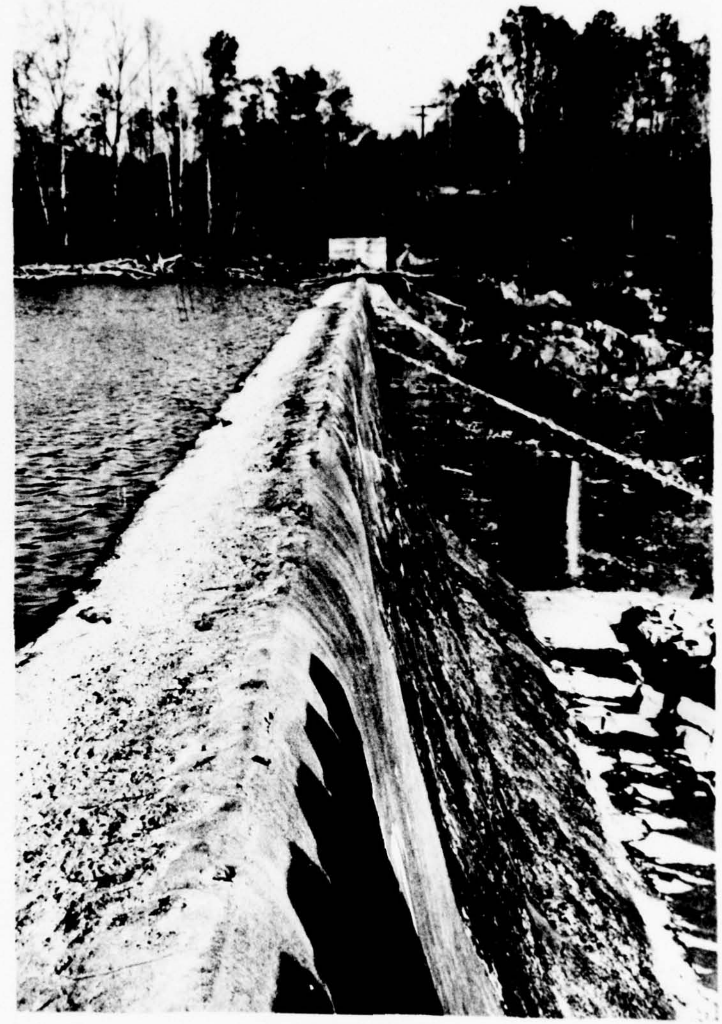
2

NORFOLK DISTRICT FILE NO.

APPENDIX II

PHOTOGRAPHS





MEHERRIN RIVER DAM IN 1963

PLATE 2

DEBRIS

VIEW OF DAM FROM LEFT ABUTMENT

PLATE 3

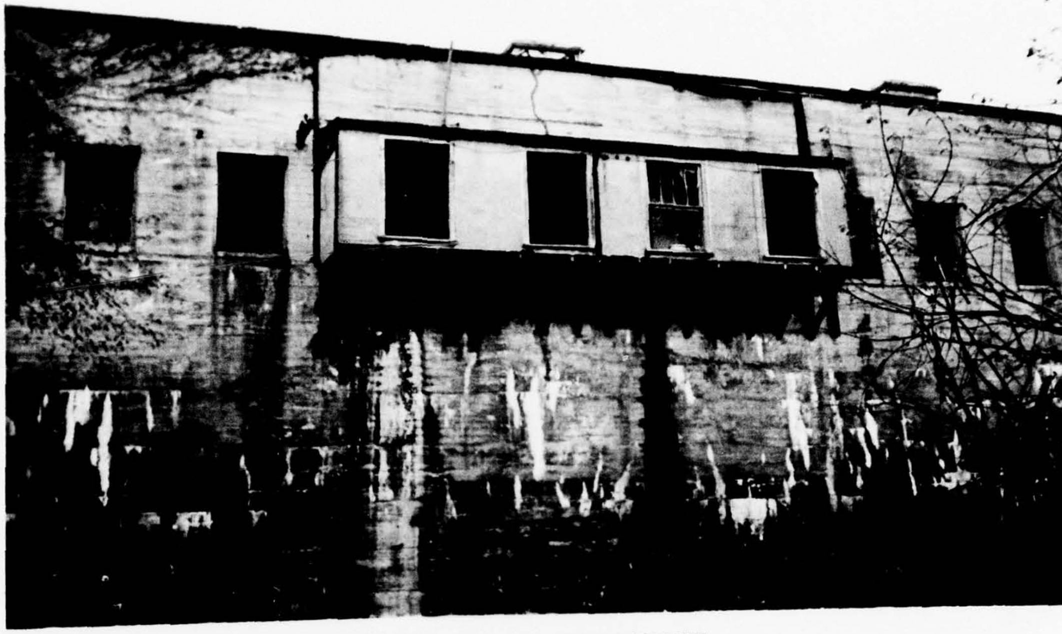
VIEW DOWNSTREAM FROM RIGHT ABUTMENT

EROSION DOWNSTREAM LEFT ABUTMENT

PLATE 4

RIGHT ABUTMENT DOWNSTREAM OF POWERHOUSE

RIGHT ABUTMENT



DOWNSTREAM OLD POWERHOUSE

UPSTREAM OLD POWERHOUSE

PLATE 6

APPENDIX III

FIELD OBSERVATIONS

FIELD OBSERVATIONS

PHASE I - Field Inspection

Name of Dam: Meherrin River Dam at Emporia (VA08101)

County: Greenville State: Virginia

Coordinates: Lat: 3641.8 Long: 7733.5

Date of Inspection: 19 April 1978

Weather: Partly Cloudy Temperature: 60°F

Pool Elevation at Time of Inspection: 113 msl

Tailwater at Time of Inspection: 74 msl

Norfolk District Inspection Personnel:

Larry Holland, Hydrologist
Jeff Irving, Structural Engineer
Ed Strawsnyder, Geotectnical Engineer (Recorder)
Leonard Jones, Engineering Technician

Other Attendees:

Bob Gay, Virginia State Water Control Board
Keith Drohan, Virginia State Water Control Board
Mr. McCord, City Manager for Emporia

1. General: At the time of the inspection, approximately 1 foot of water was flowing over the ungated spillway. This flow limited the visual inspection of the dam to the non-overflow sections and an area on the overflow section downstream of and adjacent to the left abutment.

2. Concrete Dam.

2.1 Seepage: Any observation of seepage or leakage was obscurred by the flows over the dam. Past inspections conducted by the Norfolk District in 1963 and Wiley and Wilson, Inc. in 1974 indicated numerous leaks through the dam and only one visible leak in the foundation of the dam. The foundation leak was located just north of the existing fish ladder. The leaks through the dam ranged from 5 to 10 gpm, whereas; the foundation leak was estimated at 1 cfs. As a result of the Wiley and Wilson, Inc. observations an extensive grouting program was performed in February 1977 to seal the seepage paths. The success of the program should be observed during a period of low flows when access can be gained to the downstream toe and any seepage will be visible.

2.2 Structure to Abutment Contact.

2.2.1 Left Abutment: The dam on the left abutment rises from the overflow section at elevation 112.4 to a non-overflow section at approximately elevation 120.2. The non-overflow section extends approximately 30 feet into the left abutment. The surface of the concrete in this section is relatively free from the deterioration noted on the overflow section of the dam. A review of recent borings indicated that this section of the dam bears on a moderately weathered schist. However, an inspection of the downstream rock outcrops indicates that the schist occurs in zones within the diorite. These zones range from a few inches to several feet and dip steeply. A retaining wall extends downstream from the non-overflow section. The wall has been badly deteriorated by the flows over the spillway. The abutment behind the wall is being eroded by surface runoff. This erosion will continue unless the problem is arrested by proper channelization of surface runoff. Log debris is being trapped behind the retaining wall and between the rock outcrops adjacent to the downstream slopes of the left abutment.

2.2.2 Right Abutment: The non-overflow section on the right abutment exists at elevation 117.3. This portion of the dam was raised to the existing elevation by adding a parapet wall on the original section between the powerhouse and the left abutment, a distance of approximately 140 feet. This parapet wall is pitted and spalled. Recent borings indicate that this portion of the dam is founded on a hard, competent diorite. Inspection of the outcrops downstream of this section confirmed these findings. The downstream face of the non-overflow section was cracked, spalled and partially

covered by vegetation. Evidence of the 1977 grouting efforts were apparent on the crest of the original section and on the downstream face.

2.3 Drains: No drains were observed during the inspection. Discussions with Mr. McCord revealed that drain holes were drilled in the power house to drain rain water from the upper levels to the outlet chambers. However, as a result of vandalism and miscellaneous trash, the drains have been blocked. This blockage has resulted in the flooding of the upper levels of the power house by rain water.

2.4 Water Passages: Besides the ungated spillway, the water passages are limited to the five intake or penstocks into the power house. Three of the intakes are closed off by gates. The two remaining intakes are open but the flows are restricted by the old turbines whose vanes are locked in a closed position. The other three turbines are also closed.

2.5 Foundation: Access to the foundation was limited to the downstream areas adjacent to the left and right abutments. The rock outcrops on the right abutment were predominantly a dark gray hornblende diorite. The massive outcrops in this area were angular and showed little sign of weathering. The rock downstream and adjacent to the left abutment was generally composed of the diorite with isolated zones of chlorite schist dipping steeply and striking approximately perpendicular to the dam. The exposed surfaces of the diorite have been smoothed by the flows over the dam. Visual observations indicated that the schist has been moderately to highly weathered where exposed to direct contact with the flows over the spillway. Explorations performed in 1977 on the foundation under the dam indicated that the schist zones ranged from moderately to highly weathered. This investigation also indicated that the dam and rock contact was severely deteriorated. An extensive grouting program was accomplished in February 1977 to correct this deterioration.

2.6 Concrete Surfaces: The visible portions of the concrete surfaces were severely deteriorated with numerous, irregularly shaped spalled areas highlighting the surface. These spalled areas ranged in depth from 6 to 12 inches. The exposed aggregate within these voids was angular and hard. Cracking was also apparent throughout the exposed face of the dam. The cracking was generally confined to the joints between what are believed to be the original concrete lifts. Despite the fact that the visual examination was limited, it is believed that the spalling and cracking is typical for the entire dam. These observations were highlighted in the Norfolk District's inspection in 1963 (See Drawing I, Appendix II) and again in 1974 by Wiley and Wilson, Inc.

2.7 Structural Cracking: Because of the severe deterioration of the concrete surfaces and the flows over the spillway, it was impossible to visually determine whether the observed cracking was superficial or structural. Seepage observations from previous inspections indicate that the cracks did extend through the dam. However, it is believed that these cracks and seepage paths are confined to the joints between the concrete lifts.

2.8 Vertical and Horizontal Alignment: Numerous surface irregularities were evident along the crest of the overflow spillway. Mr. McCord stated that these irregularities are the result of deterioration of the crest and range in depth from 6 and 18 inches.

2.9 Construction Joints: The severe deterioration of the downstream face of the dam highlighted the original concrete lifts. No well defined construction joints were noted during the inspection.

3. UNGATED SPILLWAY

3.1 Description: The ungated spillway or overflow section of the dam is approximately 462 feet in length and runs between the power house and the left abutment. The spillway section was raised approximately 5 feet prior to 1940. Conversation between a representative of Wiley and Wilson, Inc. and Mr. James E. Cranfill, a former power plant employee, set the date at 1913. The exposed concrete surfaces on the crest and downstream face are severly deteriorated as previously described. A fish ladder was constructed in the middle of the spillway. However, the raising of the dam negated the usefulness of the ladder. The concrete ladder is in poor condition but its condition does not affect the structural integrity of the dam.

3.2 Discharge Channel: The spillway discharges on an irregular bedrock foundation. Inspection of the foundation was limited by the tailwater.

4. POWER HOUSE

4.1 Description: The power house is located approximately 140 feet from the right abutment. The structure has not suffered the same concrete deterioration as the overflow section of the dam. However, the exterior of the power house has not been maintained as evidenced by a delapidated access bridge, rusted doors, broken windows and deteriorated stairways. The interior was not inspected because of flooding conditions caused by rainwater. Mr. McCord indicated that the abandoned equipment within the power house is not operable.

4.2 Intake: As previously described in paragraph 2.4, the five intakes or penstocks are closed off by either gates or closed turbines. The machinery associated with the operation of the gates appears to be in poor condition. A wood deck covering the openings above the turbines was rotted and should be removed. Sedimentation deposited on this deck indicated that high waters frequently rise above the intake gates.

4.3 Remarks: Conversations with Mr. McCord revealed that the City of Emporia has applied for a permit to operate the dam to produce electricity. The city also has a plan to remove two turbines and install two regulating gates. These gates would provide a means of drawing down the reservoir and controlling the silt level behind the dam. Funds have not been obtained for the installation of the gates.

5. RESERVOIR

5.1 Description: Inspection of the reservoir was limited to the areas immediately adjacent to the dam and approximately 1/4 mile upstream. A review of U.S.G.S. quad sheets and the field observations indicated that the areas bounding the reservoir are wooded with gradual slopes (less than 1v to 5h). The reservoir extends a distance of approximately 2 miles upstream of the dam. Slopes within this range are estimated at 5 feet per mile of reservoir.

5.2 Sedimentation: Sedimentation surveys conducted in 1973 indicate that the dam has silted to within 11 feet of the top of the spillway. Silting was also noted in the reservoir areas adjacent to the water treatment plant. This problem will continue unless plans of controlling the sediment are implemented.

5.3 Debris: Debris in the form of logs and tree limbs are being trapped on the upstream face of the overflow section in an area adjacent to the left abutment. Photographs taken in 1963 by the Norfolk District indicated that this problem also existed at that time. Mr. McCord indicated that log removal is a continuous and costly maintenance problem. Flows are being impeded by these log jams.

6. DOWNSTREAM CHANNEL

6.1 Condition: Besides the rock outcrops, the channel immediately downstream of the spillway is relatively free from obstructions. Log debris is presently being trapped by the outcrops adjacent to the left abutment but the flow is unobstructed. The channel narrows from 462 feet at the base of the spillway to approximately 100 feet at a distance of approximately 300 feet from the dam. The overbank areas beyond this point are heavily wooded.

6.2 Slopes: The channel immediately downstream of the dam slopes at a rate of approximately 0.15 percent. The slope reduces to 0.075 percent as the river passes through Emporia.

6.3 Downstream Inhabitants: The dam is located approximately 1 mile from the nearest inhabitated area that could possibly be affected by a failure. This estimate excludes Interstate 95 which is less than one-half of a mile downstream of the dam. It is estimated that the potentially affected area will include approximately 10 houses, an armory, a school, and 2 small businesses. This estimate is based on a visual examination of the downstream area and the use of a U.S.G.S. quad sheet. The actual affects of flooding caused by a failure of the dam cannot be determined without a detailed analysis of the downstream area.

APPENDIX IV

GEOLOGY AND REMEDIAL TREATMENT REPORTS

GEOTECHNICS, INC.

Telephone
(703) 344-4569

321 Walnut Avenue
Vinton, Virginia 24179

GEOLOGICAL EVALUATION
CORE BORINGS AND LABORATORY TESTING
MEHERRIN RIVER DAM
EMPORIA, VIRGINIA

BORING LOGS HAVE BEEN REMOVED FROM THIS REPORT
AND ARE ON FILE IN THE NORFOLK DISTRICT

Geotechnics
Commission No. 1230
4 October 1976

REPORT # 1

Comm. No. 1230 4 October 1976

GEOLOGICAL EVALUATION

CORE BORINGS AND LABORATORY TESTING

MEHERRIN RIVER DAM

EMPORIA, VIRGINIA

1. Scope - In accordance with the Contract Documents and your letter of authorization (dated 20 July 1976), we have provided geological services for Division I, Stage 1 of the Meherrin River Project (W. & W. Comm. No. 5189). We are submitting herewith our report on the investigation and evaluation of Division I, Stage 1.

2. Procedure - Eight (8) core borings were made between 24 August and 14 September 1976. All borings were made using NX bits and a double-tube, M-type core barrel. A single-tube core barrel was utilized in the upper few feet of hole to develop sufficient hole depth to use the longer double-tube barrel. Two (2) barges were employed in the work; one barge contained a Sprague and Henwood Model 40-C diamond core drill which was gasoline powered and the other barge contained a modified 40-C diamond core drill which was driven with an air motor. Both barges had overhanging support sections to permit positioning directly over the top of the dam. Accessory

equipment included a supply barge, two (2) power boats, and assorted land based equipment including an air compressor and stand-by compressor, front-end loader and fork-lift truck.

Initial plans were to drill seven (7) core borings. Five (5) borings were completed without incident. Boring No. 2 was drilled to rock where it encountered broken material. It was gravity grouted, allowed to set several days and redrilled to a satisfactory completion depth. Boring No. 5 was drilled to rock where severely broken materials were also encountered. The hole was completed to the desired depth but the tools and core were lost in the hole upon attempting final retrieval. Several recovery attempts failed after which Boring No. 5 was abandoned. Boring No. 5A was then drilled to rock one (1) foot north of the original hole, gravity grouted and redrilled to the required depth without difficulty. Boring No. 5A was subsequently pressure grouted and Boring No. 5 was gravity grouted with the lost tools in the hole.

3. General Geology - The site of the Meherrin River dam is on the Fall Line. Eastward one finds bedrock beneath an increasingly thick wedge of unconsolidated sediments which include sand, silt, clay and gravel. Westward one finds a variety of metamorphic and igneous rocks usually

capped by a veneer of sediments on the higher ground while bedrock periodically outcrops in the major stream channels. All evidence of sediments (except in stream channels) usually disappears within twelve to fourteen miles to the west of Emporia.

Since the original dam site straddles the major drainage course in the area, outcrops of bedrock appear in abundance, particularly downstream below the dam. Previous geologic literature has described the bedrock as follows: primarily hornblende gabbro and gneiss which includes amphibole chlorite schist, chloritic hornblende gneiss, some amphibolite chloritic diorite, hornblende diorite and kyanite schist and kyanite quartzite. Numerous additional related varieties of the above rock occur in lesser amounts throughout the area.

4. Site Geology - The configuration of the bedrock surface as indicated by the test borings, shows that the deepest portion of the old river channel was in the vicinity of Boring No. 1, that is, near the abandoned power house. A slight rise in the bedrock surface is noted at Borings No. 3 and 4 where the bedrock surface was some 4 to 5 feet higher. Proceeding northward, we find that the area between Borings No. 5 and 6 constitutes another low area probably representing a former river channel. It is of intermediate height between the low area at Boring No. 1

and the higher area in the vicinity of Borings No. 3 and 4.

A former river channel between Borings No. 5A and 6 would partially explain the weathered and broken nature of the bedrock in this area.

Bedrock types encountered at the dam site included (in order of abundance) hornblende diorite, chlorite schist, chloritic hornblende diorite, diabase and chloritic diorite.

5. Concrete Condition - Concrete in the dam is divided into two well-defined sections: the original dam, and a five to six foot cap or section added some years after the original dam had been constructed. The contact or joint between the old and new portions of the dam ranged in depth (below the top of the dam) from 5.2 feet in Boring No. 7 to 5.9 feet in Boring No. 6.

The concrete in the upper, more-recent section of the dam ranged from partially to highly weathered; was white to slightly gray in color; generally chalky in appearance; moderately to highly friable; usually broken and fractured; and contained pits and vugs, with some occasional alteration around the periphery of the aggregate particles. Aggregate pieces consisted of granite, diorite, granodiorite, diabase and quartz. The aggregate fragments appeared to be sound

and generally ranged up to approximately 2-inches in maximum dimension. Reinforcing steel was encountered in Borings No. 5 and 5A at 2.0 and 3.0 feet, respectively. The steel was loose in the concrete and the concrete broken in both borings. Minor rusting was observed but no significant loss of steel cross-section was noted. Compressive strengths in the capping concrete ranged from a low of 1974 psi in Boring 5A to 4466 psi in Boring No. 7.

The concrete in the original dam beneath the cap consists of partially to highly weathered white to gray concrete. The aggregate consists of granite, diabase, diorite, greenstone and schist, ranging up to 10-inches in maximum dimension. The concrete has numerous highly fractured zones, vugs, weathering around aggregate particles and some aggregate particles display reaction rims (indicative of alkali-aggregate reaction). Compressive strengths ranged from 781 psi in Boring No. 5 to 7355 psi in Boring No. 4.

6. Contact Zone Condition - The contact between the concrete dam and the underlying bedrock was significantly altered or weathered. Generally the concrete and bedrock were both altered to some degree. In some borings the concrete was more highly weathered than the rock and in others the situation was reversed. The weathered zone ranged

from 2.7 feet in Boring No. 1 (40.8 to 43.5 feet below the top of dam with the top of rock at 41.8 feet) to 31.3 feet thick in Boring No. 5A (21.0 to 52.3 feet below the top of dam with the top of rock at 40.0 feet).

7. Bedrock Condition - Bedrock at the site consists largely of hornblende diorite, chlorite schist, chloritic hornblende diorite, and diabase. Minor amounts of chloritic diorite are noted on the boring logs.

The hornblende diorite was encountered in Borings No. 1, 3, 6, and 7 and consisted of gray to bluish gray, medium-grained diorite. Epidote was noted as an accessory mineral occurring in veins and scattered single grains. The rock was usually slightly weathered and slightly to moderately fractured with some highly fractured zones.

Chlorite schist was identified in Borings No. 1, 5, 5A, and 7. In Borings No. 1 and 7 the schist occurred as bands between zones of hornblende diorite and/or chloritic hornblende diorite. The chlorite schist was the only rock type encountered in Borings No. 5 and 5A. Calcareous seams, vein fillings, fracture healings and minor amounts of pyrite were noted as accessories. In Boring No. 5 the rock was highly weathered and fractured from 40.0 feet to 45.7 feet and core recovery was low (ranging from 36 to 67 percent in short runs). Below 45.7 feet (45.7'-55.5') core recovery increased to approximately 99%. In Boring

No. 5A the rock and the immediately overlying concrete, as mentioned above, were more severely weathered and altered. The rock was highly weathered and some pieces could be crushed or crumbled by hand (particularly that from depths of 40.0 feet to 52.3 feet). The calcareous material was largely leached-out in this zone and the rock was very vuggy with pronounced weathering along the foliation planes.

Chloritic hornblende diorite was encountered in Borings No. 3 and 4. The rock was generally slightly weathered and ranged in color from bluish-gray to gray-green in color. Epidote, pyrite and calcareous materials were noted as accessory minerals. The rock was slightly to moderately fractured.

Diabase was encountered in Boring No. 2. The rock was only slightly weathered and dark gray in color. The first 10 feet of rock was highly fractured and calcareous veins and joint fillings were noted sporadically throughout.

Compressive strengths of rock were variable from boring to boring and by rock type. Compressive strengths measured in hornblende diorite ranged from 12,970 psi in Boring No. 7 to 31,580 psi in Boring No. 6. Compressive strengths of the chloritic hornblende diorite ranged from

10,422 psi in Boring No. 1 to 31,298 psi in Boring No. 4. Diabase in Boring No. 2 had compressive strengths ranging from 5070 psi to 24,496 psi. Compressive strength in the chlorite schist, which included the poorest rock material encountered, ranged from 1401 psi in Boring No. 5A to 23,967 psi in Boring No. 5.

8. Discussion - There are several general criteria which indicate a deterioration of concrete. These criteria include microfractures, presence of calcium hydroxide, carbonation of the cement paste, presence of calcium sulfoaluminate, and clarified reaction rims. Although no petrographic work was performed on the cores extracted for this project, a close examination of the cores with a 10X hand lens indicates that some or possibly all of the above general criteria are present in concrete taken from this dam. This information when considered together with the variations in core recovery and unconfined compressive strengths indicate that substantial alteration or deterioration of the concrete has occurred since the dam was constructed.

Freeze-thaw action may have been the single greatest contributor to the present condition of the dam with some evidence of, at least, minor alkali-aggregate reaction, particularly involving some of the coarse

aggregate pieces.

The numerous fractured zones, friability of the concrete, evidence of carbonation (lighter color and chalky appearance) and apparent leaching of the concrete as indicated by the presence of voids and void fillings all tend to support the conclusion of gradual but persistent deterioration of the concrete in the dam. The deterioration is not expected to abate or decrease, but is likely to continue at an ever increasing rate.

We strongly recommend grouting of the dam as one method of sealing or partially sealing construction joints, fractures, and possibly some voids within the concrete to reduce the permeability and prevent the access of surface waters to the interior concrete. Moreover, grouting will decrease leakage and increase stability in the contact zone. While it is doubtful that any remedial measures will halt deterioration at this stage, we believe that grouting will certainly retard deterioration and add to the useful life of the structure.

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TABLE I

Boring No.	LOCATION Station	Elev.	Unconfined Compression (psi)	Core Recovery (%)	Remarks
1	0+40.8	109.1 (s)	3563	85	Concrete
		105.9	2169	94	Concrete
		105.0	2814	94	Concrete
		96.0	2817	99	Concrete
		86.5	1972	99	Concrete
		65.3	15775	95	Rock
		59.6	9014	97	Rock
		55.1	10422	97	Rock
		50.4	20845	99	Rock
2	1+00.8	109.9	2028	80	Concrete
		104.5 (s)	2042	98	Concrete
		103.9 (s)	2056	98	Concrete
		100.6	1718	98	Concrete
		86.0	1437	94	Concrete
		64.1 (s)	6479	74	Rock
		61.7	5070	99	Rock
		56.3	24496	99	Rock
		50.9	16377	99	Rock
3	1+60.8	106.1	2401	82	Concrete
		99.0	2904	99	Concrete
		85.6	3158	87	Concrete
		68.7	29887	96	Rock
		63.4	27663	96	Rock
		58.5	17200	88	Rock
		53.3	12125	88	Rock

TABLE I

Boring No.	LOCATION Station	Elev.	Unconfined Compression (psi)	Core Recovery (%)	Remarks
4	2+20.8	106.1	4173	93	Concrete
		104.9	7355	93	Concrete
		98.4	1591	99	Concrete
		86.6	2444	96	Concrete
		68.4	27633	99	Rock
		63.4	18610	99	Rock
		58.5	31298	99	Rock
		52.9	19737	98	Rock
5	2+80.8	106.5	4173	96	Concrete
		99.8	2791	99	Concrete
		88.5	781	99	Concrete
		64.8 (s)	9305	97	Rock
		61.7	23967	97	Rock
5A	2+81.8	109.7 (s)	1974	77	Concrete
		107.0	3976	90	Concrete
		105.7	3918	90	Concrete
		100.0	2538	98	Concrete
		88.5	1592	99	Concrete
		58.3 (s)	1401	89	Rock
		52.8 (s)	20022	97	Rock
6	3+40.8	110.0	4173	76	Concrete
		106.7	4004	99	Concrete
		105.6	4060	99	Concrete
		97.0	3835	99	Concrete
		87.8	1748	99	Concrete

TABLE I

Boring No.	LOCATION Station	Elev.	Unconfined Compression (psi)	Core Recovery (%)	Remarks
6	3+40.8	66.3	31580	84	Rock
		61.2	21993	98	Rock
		56.3	18320	98	Rock
		52.0	14662	99	Rock
7	4+00.8	109.1	4466 (s)	78	Concrete
		106.2	4018	98	Concrete
		105.6	5443	98	Concrete
		95.5	3158	94	Concrete
		87.0	1664	96	Concrete
		79.7	17764	97	Rock
		74.2	13534	99	Rock
		69.7	12970	99	Rock
		64.7	17200	99	Rock

NOTES:

1. North face of Power House Station 0+00.
2. The designation (s) means "substitute" and indicates the sample was not suitable at the specified depth, and another sample was substituted from as near as possible to the specified depth.

   Due to the poor condition of some areas of rock and concrete certain samples were unobtainable. The depth where no sample was tested are listed as follows:

NOTES: cont.

Boring 3 - Station 1+60.8 - 0'-1' and 5'-6' depths

Boring 4 - Station 2+20.8 - 0'-1' depth

Boring 5 - Station 2+80.8 - 0'-1' and 5'-6- depths

14'-15' and 19'-20' rock depths

Boring 5A-Station 2+81.8 - 4'5' and 9'-10' rock depths

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CORE DRILLING, GROUTING, AND ROCK ANCHOR INSTALLATION
MEHERRIN RIVER DAM
EMPORIA, VIRGINIA

BORING LOGS HAVE BEEN REMOVED FROM THIS REPORT
AND ARE ON FILE IN THE NORFOLK DISTRICT

Geotechnics
Commission No. 1230
7 April 1977

REPORT # 2

Comm. No. 1230 7 April 1977

CORE DRILLING, GROUTING, AND ROCK ANCHOR INSTALLATION

MEHERRIN RIVER DAM

EMPORIA, VIRGINIA

1. Scope - We are submitting herewith our final report on the above identified project Division I Stage 2, Stage 3, and post-Stage 3 drilling and grouting. A previous report was submitted on Division I, Stage 1 of this report.

2. Procedure - Division I Stage 2

Stage 2 of Division I consisted of redrilling the seven (7) original holes with percussion tools and installing seven (7) rock anchors. The percussion drilling was accomplished between 20 September and 1 October 1976. The anchors or bolts were placed in the holes, torqued, tensioned and left for at least 48 hours before testing. At the end of the waiting period the bolts were tested and those having relaxed by 5000 pounds or less were grouted. Beginning with Anchor (Bolt) No. 7 an attempt was made to set the anchor plate and bolt below the crest of the dam, as specified in the Project Manual. Extreme difficulty in working below the

surface of the dam resulted in a request from the contractor, and approved by Wiley and Wilson, to recess only the top washer (anchor plate) into the dam. This facilitated anchor installation by allowing common hand tools, rather than specialized tools, to be used on the locking nut and allowed better visibility of the anchor plate and nut. Of the thirty (30) anchor bolts installed, only Anchor No. 7 (located $\pm$ 60' south of the North Abutment) is recessed below the top surface of the dam. When the anchors were tested, all were approved with the exception of Anchor No. 4, which had relaxed by 5400 pounds. Anchor No. 4 was retorqued, retensioned and tested again. This test indicated relaxation of only 200 pounds and the anchor was accepted.

All anchors were grouted satisfactorily with the exception of Anchor No. 7 which took only $\pm$ $\frac{1}{2}$ CF of grout at $\pm$ 160 psi pressure. As was the case with several anchors in Stage 3 (to be discussed later) the contractor was able to pump air thru the anchor rod, but was unable to pump water thru the rod. No return of pumped grout was noted at the top of the anchor. The grout return tube of Anchor No.

7 was pinched and therefore unable to accept grout. Three (3) extra washers (total of 4) had been added to Anchor No. 7 between the locking nut and the base plate to overcome a problem of fouled threads on the bolt. These additional washers and the recessed location of the anchor plate limited visibility and accessibility to the work and resulted in the inadvertent damage to the grout return tube noted above.

Stage 2 was completed (final grouting of anchors) on 26 October 1976. All anchors (Nos. 1 thru 7) were installed without major problems, with the exceptions of Nos. 4 and 7 as discussed above.

3. Procedure - Division I Stage 3

Stage 3 of Division I consisted of core drilling, grouting, percussion drilling, and the installation of twenty-three (23) additional anchor bolts. Core drilling for Stage 3 began on 18 October 1976. Traces of the earlier placed grout were identified in some concrete and rock cores, indicating grout from Stage 1 had been pumped laterally thru the dam a minimum distance of roughly 30 feet in a few instances. Borings in this stage also helped define problem zones (i.e., leaks in the dam or the highly weathered contact zone) which could be treated by

additional grouting after the completion of Stage 3. Borings which collapsed or proved difficult to drill were reduced from NX to BX size. This method proved more efficient and effective than the gravity grouting and accompanying waiting period which slightly delayed the completion of the Stage 1 drilling.

Examination of the rock core was the basis for determining minimum depths at which anchors could be set. The depths were adjusted up or down so that wherever possible adjacent anchors would not be set at identical depths. In a few instances, adjacent anchors were located at equal depths because of other considerations.

Grouting of the Stage 3 drill holes emphasized two (2) important findings:

(1) Average grout-take per hole was still increasing with each series of holes drilled across the dam. This was discussed in Geotechnics letter dated 12 January 1977.

(2) Significant observable leaks on the downstream face of the dam were limited to the area surrounding Borings No. 6, 13 and 26.

During the grouting of the Stage 3 core holes, grout was observed flowing over the dam. Initially this was interpreted as indicating a leak high up on the upstream face of the dam. Difficulty

encountered in stopping these "apparent leaks" led to utilization of a packer in conjunction with the standard grout pipe. The grout hole was filled with grout from the bottom up, then the grout pipe was pulled and the packer was set roughly two (2) feet below the top of the hole. With this procedure the "apparent leaks" stopped and pressure could be maintained in the hole. During this period of the work, water over the dam was running high (more than 4 inches deep) and work areas were difficult to dewater. After several dewatering attempts met with only minor success, the holes were grouted without dewatering. Sometime later in Stage 3 when the final few anchors were being installed the above mentioned "leaks" were observed under better conditions and reinterpreted. The situation was found to result from grout leaking around the grout pipe (or otherwise coming out at the top of the hole), and being denser than the water, flowed in an upstream direction down the sloping crest of the dam and was caught up in the rising current of water flowing over the dam. The outward appearance was that of an extensive leak (at times up to 10 feet wide) across the top of the dam. The final appraisal of the situation

is that most of these "apparent leaks" were not in the dam, but rather due to inadequate caulking around the grout pipe. A few actual leaks were observed or inferred on the upstream side of the dam but these were either minor in nature or appeared to be stopped during the course of grouting. Core drilling and grouting for Stage 3 were completed on 6 December 1976. Grouting for Stage 1 and Stage 3 is summarized in Table I.

Percussion drilling for Stage 3 began on 23 November 1977. Five (5) borings (Borings No. 9, 10, 20, 23 and 25) collapsed during drilling operations or before anchors could be installed. All were regrouted. Borings in which either heavy reinforcing steel , vertical steel, or other unusual conditions were encountered were partially drilled with coring tools in order to save time and avoid excessive damage to the percussion tools. Percussion drilling was completed on 2 February 1977.

Anchor bolt installation for Stage 3 began on 15 December 1976. Anchors installed in Stage 3 were torqued, and loaded to $\pm$ 65 tons. The load was held for at least 10 minutes. If the load relaxed by 5000 pounds or less the anchor rod was locked-off and grouted. If the load loss was in excess of

5000 pounds, a second load test was made or the anchor was retorqued and retested. See Table II.

Of the twenty-three (23) anchor bolts set in Stage 3, eight (8) were grouted normally (i.e., took approximately the estimated amount of grout thru the rod or thru the rod and grout tube combined; or took an amount of grout (2 CF) agreed to by the bolt manufacturer, Wiley and Wilson and Geotechnics to be the minimum acceptable).

Four (4) anchor bolts had takes of less than 2 CF (ranging from 0 to 1½ CF) and were capped before serious concern was voiced regarding the grout-take of the anchor bolts. Three (3) anchor bolts had takes of less than 2 CF (ranging from 0 to 1 CF) and were not capped until a final attempt was made to grout thru the grout tube. Eight (8) anchor bolts were grouted thru the rod prior to torquing. This method resulted in satisfactory grout takes for the anchor bolts but caused some minor problems in torquing and setting. Apparently grout trapped between the anchor and the wall of the hole acted as a lubricant and inhibited the initial anchoring attempt. Generally greater amounts of slippage, during initial tensioning,

occurred in the pregrouted holes. If slippage was judged to be excessive the anchor was retorqued and reloaded. Holding tests and lock-off at the prescribed load range were conducted as usual. The only difficulty experienced was in the initial loading. All anchor bolts installed after 6 January 1977 were grouted prior to torquing. It was understood that an attempt would be make to devise a method of grouting thru the anchor plate for those anchors which would take no grout thru the rod or grout tube. In some instances neither water nor air could be pumped thru the rod and air thru the grout tube bubbled out around the grout tube. On 24 January 1977, a 3/4-inch hole was drilled thru one anchor plate on an installed anchor and a ½-inch copper tube was inserted thru the hole. When the tube was inserted perpendicular to the anchor plate the end of the tube stopped on concrete outside the original drill hole. When the tube was slanted to miss the concrete the end of the tube was stopped by the rod itself. The intent of the experiment was to attempt to get the copper tubing (in 10-foot sections with thin-walled couplings) as deep as possible into the hole around the anchor

rod. When maximum depth was reached grout was to be pumped thru the tubing thereby providing some assurances that grout would reach the zone of anchorage. Had this method proved successful, the caps would have been removed from the four (4) anchor bolts with short takes which were already capped, and these anchors would have been grouted by this method. Since the experimental method failed, the caps were not disturbed.

The three (3) anchors with short grout takes that had not been capped were reserved until completion of the project. At that time a final attempt was made to grout these anchors. In all three instances no grout could be pumped thru the rod. Attempts to grout thru the grout tube met with negligible results. Takes ranged from an estimated ½ CF in two (2) of the anchors to an estimated 1 CF in the third. The grout leaked out around the grout tube in all instances after a very brief period of pumping, and the take in the anchor (No. 28) where 1 CF was recorded is probably overestimated. These three (3) anchors were capped upon completion of the final grouting attempt.

Rock anchor bolt installation was completed

on 4 February 1977 and the final grouting attempt and capping was completed on 4 March 1977.

4. Procedure - Post-Stage 3 Grouting Program

Core drilling and grouting during Stage 3 further defined zones of weakness in the dam, and an additional grouting program was considered. This program was formally recommended by Geotechnics' letter dated 12 January 1977. The grouting program was approved and initial core drilling began on 3 February 1977. The final program consisted of thirty (30) core drilled grout holes: two (2) holes in the north abutment area, three (3) holes in the south abutment area, and twenty-five (25) holes across the dam proper. The procedure involved drilling several holes spaced across the dam, grouting those, and based upon grouting results, selecting another group of locations from the original plan to be drilled and grouted. In the abutment areas,holes were drilled as more of an exploratory program than was the case on the dam, since no previous work had been done in the abutment areas to determine potential or existing zones of weak rock or concrete. A summary of grout takes is attached as Table III.

Locations were selected for grout holes based upon previous grout takes and the condition of drill cores from Stage 1 and Stage 3 core drilling, especially in those zones with high core losses indicating highly weathered zones near the dam-bedrock contact.

It was anticipated that successive series of grouting in these areas should result in decreasing grout takes as the program progressed. Overall results were as follows:

1st Series (11 holes)	26 CF/hole (average)
2nd Series (7 holes)	24 CF/hole (average)
3rd Series (4 holes)	9 CF/hole (average)

Three (3) areas on the dam had grout takes higher than anticipated. These areas: Vicinity of Boring No. G-2; Vicinity of Borings No. G-8, G-9, and G-10; and Vicinity of Borings No. G-17, G-18, G-19, and G-20 will each be examined.

Boring No. G-2 was located between Anchor Bolts No. 15 and 30 (approximately 22.5 feet south of the North Abutment) and was the first of the borings to be grouted. The grout-take of 78 CF was considered to be quite high. No leaks were observed and the grout-take was steady at 12-15 psi pressure. The rate of take gradually slowed and

stopped. The grout system held $\pm$ 15 psi pressure. It was concluded that the grout was being pumped laterally thru the dam and North Abutment area. This conclusion was supported by significantly reduced grout-takes in adjacent borings grouted after No. G-2 and by traces of grout noted in one boring (No. N-2) located in the North Abutment area.

Borings No. G-8, G-9 and G-10 were located in the area bounded by Anchor Bolts Nos. 13 and 27. This was the area involving a sizeable leak on the downstream face approximately in the vicinity of Anchor Bolt No. 26. A leak in this zone was noted during Stage 1 when Boring No. 6 was grouted. At that time the major leak was plugged with burlap and pressure was maintained in the hole with low grout-take. Grouting of Borings No. 13 and 26 during Stage 3 also indicated a leak in essentially the same vicinity. The water level was too high during this period for a man to get to the area of the leak to plug it. In both instances grouting operations were halted before the leak was stopped. Both borings were checked later and grout levels in the holes were considerably above the level of the leak. In the

case of Borings No. G-8, G-9, and G-10, the nearest post-Stage 3 grout holes (Borings No. G-7 and G-11) each had grout-takes of 12 CF. Boring G-10 (located between Anchor Bolts No. 13 and 26) took 36 CF of grout. The take was slow at 12-15 psi pressure. No leaks were observed and the hole held at $\pm$15 psi pressure when the grout-take stopped. Boring No. G-9 (located between Anchor Bolts No. 26 and 6) was grouted immediately after No. G-10. A sizeable leak was noted in the same vicinity as that discussed above. The rate of grout-take was rapid and no pressure could be maintained. After 30 CF had been pumped into the hole, and the leak continued, the hole was abandoned temporarily. The next day the hole was checked and the grout level was at 22 feet below the top, well above the level of the observed leak. An additional 12 CF was pumped into the hole at this time, and no leak was observed. The previous day's grout, settling overnight, apparently had choked the opening from Boring No. G-9 to the leaking zone. Boring No. G-8 was grouted immediately after the initial attempt to grout No. G-9. The rate of grout-take varied but finally slowed and stopped with 30 CF at $\pm$ 15 psi pressure. No leaks were observed and the

hole held ± 9 psi pressure.

In the area of Borings No. G-17, G-18, G-19, and G-20, the order of grouting was G-18, G-19, G-17 and G-20. Boring No. G-18 took only 7 CF of grout and held ± 12 psi pressure. Boring No. G-19 was grouted immediately after G-18, and took 90 CF at pressures ranging from 6 to 12 psi. The rate of take gradually slowed down. No leaks were observed either upstream or downstream. The hole was temporarily abandoned to allow the grout to settle and set-up for a few hours. When the hole was checked several hours later the level of grout was found to be at ± 5 feet. Another 12 CF were pumped into the hole at ± 6 psi pressure and again no leaks were observed.

Boring No. G-17 was grouted immediately after the first attempt to grout G-19. Faint traces of grout indicated small leaks on the upstream side. The hole took 30 CF and held ± 15 psi pressure. The upstream leaks were essentially stopped. Boring No. G-20 was grouted several days after the above discussed borings. The hole had a grout-take of 24 CF and no leaks were observed. The hole held at ± 15 psi pressure.

At the conclusion of the on-dam grouting it was decided that two (2) additional borings would be drilled: one (1) beside G-9 to check the status of

the leak in that area and another beside G-19 to check the high grout take in that area. The two borings were drilled between Boring No. G-9 and Anchor Bolt No. 26 (G-9A) and between Boring No. G-19 and Anchor Bolt No. 19 (G-19A).

Grouting of No. G-9A disclosed a leak in the same vicinity as noted before but the leaking stopped quickly and the hole maintained a pressure of $\pm$ 15 psi. The hole was completely grouted in one attempt using only 25 CF of grout indicating considerable improvement in the zone surrounding the leak. Boring No. G-19A was grouted and took only 13 CF indicating significant improvement in that area.

Two (2) borings were drilled in the north abutment to investigate the condition of the concrete and the underlying bedrock. Borings near the north abutment encountered several feet of highly weathered material immediately below the bottom of the dam. In the two (2) abutment holes Nos. N-1 and N-2 the weathered zone was less extensive than that encountered in other borings. Grout-takes were also quite low. The concrete core taken from Boring No. N-2 indicated traces of grout from previous grouting operations.

Three (3) holes were drilled in the south abutment area upon the recommendation of Wiley and Wilson. Each hole was drilled and grouted before the next hole was drilled. Boring No. S-1 (nearest the power house) had a void from 1.0 to 1.7 feet. This void occurred under what appeared to be a concrete patch or the site of some type of previous repair work. A grouted zone was noted in Boring No. S-1 from 13.6 to 14.0 feet indicating grout from previous grouting operations had been pumped laterally over 100 feet. When the hole was grouted grout leaks were noted near the void (which was also the same location where drill water had leaked when drilled) mentioned above. This leak was effectively sealed off with a packer and grouting continued. The hole took 26 CF of grout and held at ± 15 psi pressure when the take stopped. Boring No. S-2 was drilled approximately 50 feet south of Boring No. S-1. Boring No. S-2 took only 7 CF of grout and held at ± 15 psi pressure. The two (2) holes were split-spaced by Boring No. S-3 to test and verify the effectiveness of the grouting of the other two (2) holes. Boring No. S-3 took only 7 CF of grout and held at ± 15 psi pressure, indicating the apparent success of the grouting

program in the south abutment area.

All drilling and grouting for the post-Stage 3 grouting program was completed on 4 March 1977. The contractor had personnel involved in demobilization on the site for several additional days.

TABLE I

GROUTING SUMMARY

DIVISION I, STAGES 1 AND 3

MEHERRIN RIVER DAM REPAIRS

EMPORIA, VIRGINIA

Boring Number	Cement	Fly Ash	Sand	Total (Cu. Ft.)
1	25	5	0	30
2	5	1	0	6
3	10	2	0	12
4	9	1	1	11
5a	14	2	0	16
6	10	2	1	13
7	35	6	4	45
Total Stage 1	108	19	6	133
8	22	4	2	28
9	21	4	0	25
10	28	5	0	33
11	18	3	2	23
12	18	3	2	23
13	48	9	8	65
14	15	3	2	20
15	12	2	1	15
16	23	4	0	27
17	15	3	0	18
18	20	4	0	24
19	72	14	0	86

cont. Table I

Boring Number	Cement	Fly Ash	Sand	Total (Cu. Ft.)
20	24	5	0	29
21	30	6	0	36
22	40	8	0	48
23	44	9	0	53
24	15	3	0	18
25	44	9	0	53
26	83	14	8	105
27	16	3	0	19
28	15	3	0	18
29	27	5	0	32
30	15	3	0	18
Total Stage 3	665	126	25	816
Total Stage 1	108	19	6	133
Total Stage 3	665	126	25	816
GRAND TOTAL	773	145	31	949

TABLE II

ANCHOR BOLT SUMMARY

DIVISION I, STAGE 2 AND 3

MEHERRIN RIVER DAM REPAIRS

EMPORIA, VIRGINIA

Anchor Bolt Number	Length (Ft.)	Torque (Ft.-Lbs.)	Load (Tons)	Grout Take (CF)
1	55	1280	65.7	3
2	60	1280	63.5	2 7/8
3	60	1200	63.5	3 1/8
4	60	2480	65.7	2 1/2
5a	60	1360	62.4	2 1/2
6	60	1360	64.8	2
7	45	1440	65.4	± 1/4 **
End of Stage 2				
8	45	1920	65.2	2 1/2 *
9	60	1480	67.8	1+ ±1/4 Thru grout tube on 4 March 77
10	55	1480	67.5	3 *
11	50	1400	67.7	2 1/2
12	50	1400	66.6	3
13	55	1600	66.6	4 1/2 *
14	55	1480	67.8	3 1/2 *
15	35	1560	66.5	2
16	50	1600	65.0	1/2 **
17	55	1360	64.0	1 **
18	55	1440	64.0	3
19	60	1920	65.5	3 1/2 *
20	55	1580	66.0	± 1/4 Thru grout tube on 4 March 77

cont. Table II

Anchor Bolt Number	Length (Ft.)	Torque (Ft.-Lbs.)	Load (Tons)	Grout Take (CF)
21	60	1280	65.5	1 1/2 **
22	55	1400	68.8	0 **
23	60	1400	67.3	3
24	65	1680	66.8	3 *
25	60	1720	64.8	3 *
26	55	1600	66.5	3 *
27	60	1400	66.6	3
28	60	1680	65.0	± 1 Thru grout tube on 4 March 77
29	40	1440	63.5	2
30	50	1440	67.4	2 1/2

* Bolts grouted before torquing

** Bolts capped before decision reached regarding alternate grouting procedures.

TABLE III

GROUTING SUMMARY

POST-STAGE 3 GROUTING PROGRAM

MEHERRIN RIVER DAM REPAIRS

EMPORIA, VIRGINIA

Boring Number	Cement	Fly Ash	Sand	Total (Cu. Ft.)
G-1	—	—	—	Boring Omitted
G-2	65	13	0	78
G-3	7	1	0	8
G-4	7	1	0	8
G-5	10	2	0	12
G-6	16	3	0	19
G-7	10	2	0	12
G-8	25	5	0	30
G-9	35	7	0	42
G-10	30	6	0	36
G-11	10	2	0	12
G-12	7	1	0	8
G-13	7	1	0	8
G-14	7	1	0	8
G-15	12	2	0	14
G-16	7	1	0	8
G-17	25	5	0	30
G-18	6	1	0	7
G-19	85	17	0	102
G-20	20	4	0	24
G-21	11	2	0	13
G-22	6	1	0	7
G-23	16	3	0	19

cont. Table III

Boring Number	Cement	Fly Ash	Sand	Total (Cu. Ft.)
N-1	3	1	0	4
N-2	3	0	0	3
S-1	26	5	0	31
S-2	7	1	0	8
S-3	6	1	0	7
G-9A	21	4	0	25
G-19A	11	2	0	13
TOTAL	501	95	0	596

APPENDIX V

PAST FIELD INSPECTION & PERTINENT CORRESPONDENCE

NACEN-D

Inspection of Virginia Electric & Power Co. Dam at Emporia

THRU Chiefs, Design Br; Engrg Div   Chief, M&S Sec   4 Dec 63

TO Chief, Planning & Reports Br

1. On 15 November 1963 the undersigned in company with Mr. Fred E. Peele visited the VEPCO dam on the Meherrin River at Emporia, Va, for the purpose of inspecting the dam to determine its physical condition and vulnerability during a flood in the magnitude of the 1940 flood. We were met at the powerhouse by Mr. C. R. King, the plant Supt. Previously, Mr. J. A. Rawls, Manager, Engineering and Construction, Richmond, had sent drawings for the dam.

2. The original drawings indicated a dam with a crest elevation of 108 feet. The dam was built by the Greenville Water Power Co. in 1908 and acquired by the Virginia Public Service Co. at some later date. The concrete was made of crushed granite of apparently good quality. At some time prior to 1940 the dam was raised to a crest elevation of 112.4. At the same time a parapet wall was added to the non-overflow section on the right bank raising that section to an elevation of 117.3. On the left bank the abutment is 112.4 + 7.8 = 120.2 feet. The intake and forebay section is open and is planked over. Its elevation is only about a foot above the present crest of the dam, so that it is often flooded. In this section the concrete wall of the powerhouse forms part of the dam.

3. The plant is presently operated as a semi-automatic plant and is visited once a week by personnel from Roanoke Rapids. The plant has not been in operation much lately because of lack of water. At the present time, the water was about 1 foot below the crest of the dam and was holding constant, the normal river flow just about matching the leakage thru and under the dam and thru the draft tubes.

4. The crest of the dam showed signs of deterioration in places, with some pockets 3 or 4 inches deep in the surface, although the controlling elevation has been maintained by patching. The downstream face of the dam is pock marked with holes, usually quite jagged and irregular in shape and depth. Some potholes were 12 to 18" deep. Many had started at lift lines in the original pouring, and appeared to be the result of cavitation rather than deterioration. The exposed aggregate appears hard, sharp and clean.

5. A concrete fish ladder at the center of the spillway does show marked signs of deterioration of the concrete and it is questionable if it ever served any useful purpose except to buttress the dam.

6. The dam is founded on massive rock which outcrops all across the river bed and on the abutments. The outcrops, where water washed, are sound and it is assumed that the dam is placed on a similar material.

7. There is noticeable leakage at various points across the face of the dam. Most of this is minor and appears to be through lift joints and in a couple of cases through vertical monolith joints. There is some seepage under the dam, which appears as a mineral-bearing water at the toe, between the original toe and a level apron about 8' wide which apparently was placed later.

8. A section of the downstream surface about 40 feet wide immediately adjacent to the powerhouse had been resurfaced with Gunite at sometime during VEPCO ownership. This is in fair condition but is beginning to spall off.

9. Quick studies on the stability of the dam were run under two conditions.

a. Headwater to crest, no tailwater, uplift not considered. The dam was analyzed at the change of slope points on the face at El. 97, 84 and at the bottom (72).

(1) At elevation 97, the resultant force fell right on the 1/3 point.

(2) At El. 84 the resultant force was 1.6' outside the 1/3 point but 4.2' inside the face.

(3) At the bottom the resultant force was 13.9 inside the toe and 3.5' inside the middle third.

b. Headwater 8 feet above crest and tailwater 8 feet below the crest, no uplift. At the same points analyzed above the following resulted.

(1) At 97' the resultant force fell 1.8' outside the toe and 5.6' outside the middle third, indicating tension at the upstream heel of approximately 9.3 psi. This is offset by 3/4" bars at 15" c to c.

(2) At El. 84 the resultant force fell 1.3 feet inside the toe, 4.5 feet outside the middle third.

(3) At the base the resultant force fell 11.2 feet inside the toe and 0.5 feet inside the middle third.

10. A plot of a normal lower nappe curve indicates the possibility of negative pressures on the crest and downstream surfaces, and some pitting on the crest indicates this may have happened.

11. That the dam was stable for floods up to the size of the 1940 flood is obvious. How it might react to a recurrence or to a flood of greater magnitude is doubtful and might well depend on the condition of the reinforcement in the upstream face, about which nothing can be determined.

12. At any event it would seem desirable to repair the concrete surface of the dam on the crest and on the downstream face by some system such as prepakt concrete or gunite, properly applied.

13. Attached hereto are drawings received from the VEPCO on 14 Nov 63 showing the original design and present elevations. The drawing prepared in July 1940 would

indicate that date as the approximate time of transfer from Virginia Public Service Co. to VEPCO. It may be noted that the cross-section taken through the storage reservoir in 1946 showed little evidence of silting. There are large accumulations of logs and trees on the left side of the spillway and similar piles below the dam indicating the passage of very large trees and logs in times of high water.

14. A further study of the stability under the conditions of the 1940 flood considering uplift over the entire base at 100% reveals that the resultant force at the base of the dam falls only 1.05 feet inside the toe. The sliding factor is $\frac{40,000}{22,965} = 1.74$. This would indicate potential failure if the dam were on a plane surface. It is probable that failure would be prevented by the unevenness of the foundation rock and the shear resistance of the concrete.

2 Incl
1. Photos
2. Dwgs

H. D. BURT

WILEY & WILSON, INC.
LYNCHBURG, VA.

COPY

January 23, 1974

Mr. Robert K. McCord
City Manager
City of Emporia
P. O. Box 511
Emporia, Virginia 23847

RE: Dam Investigation
Comm. No. 3010

Dear Mr. McCord:

This is a report, Part I, on the condition and stability of the dam as outlined in our proposal of January 26, 1973. Structurally, the dam has deteriorated only slightly from its original condition. In checking the stability of the dam, we have found that the structure is unstable.

Our visual inspection of the dam was limited to the top and downstream face. We are assuming that the upstream face of the dam is in sound condition since it has been protected from severe weathering.

The power house appears to be structurally sound. Since the generating room was full of water, we were only able to see the structure above this level. As you know, the windows and doors are out and the wood floor is beginning to rot due to exposure to the weather. The roof of the power house appears to be sound and has only minor leaks.

We have photographed and where possible measured and located all significant cracks and large spalled areas. These defects do not seem to be critical to the dams integrity. There appears to be no interconnecting of the cracking so as to indicate structural damage. The cracks and spalls should be repaired soon to prevent any structural damage developing.

The dam has only one visible leak in its foundation. This we judge to be about one cubic foot per second. This leak should be plugged by grouting. Several other leaks exist in the cracked areas of the dam but are only small trickles of water (approximately 5 to 10 gallons per minute each.)

In checking the total dam for stability, we considered the flood crest of 5'-6" (1940 Flood) and the present day level of siltation. This yields a safety factor against overturning of only eight per cent. The accepted minimal safety factor is twenty-five per cent. Further checking of the dam at the top of the lifts at elevation 83.4' and 94.0' indicated that the dam could fail at either of these positions during a flood.

Analysis of the original dam (5'-0" lower than existing) showed that the structure would be stable with the present silt levels and a flood crest of 5'-6". Our conclusion is that the dam should never have been raised.

Our recommendations are:

(1) Lower most of the spillway 5'-0" between the power house and the north end of the dam.

(2) Maintain the silt level at the present elevation or lower in the area immediately adjacent to the upstream face of the dam.

(3) Repair the cracked and spalled concrete on the downstream face of the dam.

We will wait for your authorization before we proceed with recommendations for repairs and improvements. If you have any questions, please let us know.

Very truly yours,

WILEY & WILSON, INC.

W. B. Nolen, P.E.

WBN/jm

bcc: R. C. Dodl, Jr., P.E.

WILEY & WILSON, INC.
A PROFESSIONAL CORPORATION
ENGINEERS · ARCHITECTS · PLANNERS
LYNCHBURG - RICHMOND

2310 LANGHORNE ROAD
LYNCHBURG, VA. 24501
804-847-0192

September 10, 1974

Mr. Robert K. McCord
City Manager
City of Emporia
P. O. Box 511
Emporia, Virginia 23847

Re: Dam Investigation
Comm. No. 3010

Dear Mr. McCord:

This is a report of part II of our investigation of the Emporia Dam. The objective of this report is to:

(1) Outline repairs and improvements needed

(2) Outline a method of controlling the water level of the dam

(3) Present alternatives with a cost estimate of each

(4) Recommend the most feasible alternative

As we understand your problem you need:

(1) Improvement of the structural stability of the dam as recommended in report part I dated January 23, 1974.

(2) Some means of controlling the silt around the water intake to the cities water supply.

(3) A method of drawing down the water level of the dam to provide some natural flushing of the reservoir silt.

(4) Repairs to the concrete spillway surface.

We offer the following three schemes as possible solutions:

Scheme I (Lower 400' of Spillway 5')

To provide structural stability remove a 400 foot length of the top 5 feet of the spillway between the power house and the north abutment. The removal of the 5 foot height should begin at the power house and leave about 60 feet of spillway intact near the north abutment. Resurface the lowered spillway crest with new concrete.

Repair the downstream face of the spillway with gunite in areas where the concrete has deteriorated due to cracking and spalling. Plug the small leak in the foundation with grout (leak 140± from north abutment).

The problem with this scheme is that the lake would be lowered five feet, reduce the reservoir capacity and make the cities water intake structure almost inoperative due to present siltation. The estimated cost of Scheme I is:

Modification of 400' of Spillway	$106,500
Repair of Spillway	$ 93,500
TOTAL	$200,000

Scheme II (Stabilize Dam with Rock Anchors without Siltation Control)

To provide structural stability anchor the spillway section to bedrock with 140 kip rock anchors, spaced at 5'0" center to center along 400 feet of spillway. These anchors will require drilling vertically thru the concrete dam two feet off the upstream face and into bedrock.

As in Scheme I the downstream face of the spillway would require repair with gunite and plugging of the foundation leak.

With this scheme the siltation adjacent to the dam could be neglected and the dam may fill with silt. The drawback here is that the entire dam will continue to fill with silt at its current rate and no improvement is made to the dam as a water reservoir.

The estimated cost of Scheme II is:

Installation of Rock Anchors @ 5'-0" c/c	$223,000
Repair of Spillway	$ 93,500
TOTAL	$316,500

Mr. Robert K. McCord
September 10, 1974
Page three

Scheme III (Stabilize Dam with Rock Anchors with Siltation Control)

To provide structural stability anchor the spillway section to bedrock with 140 kip rock anchors at 15'-0" center to center along 400 feet of spillway. These anchors will require drilling vertically thru the concrete dam two feet off the upstream face and into bedrock. Also, this scheme is predicated on the present silt level adjacent to the dam being maintained (i.e. the silt will not be allowed to accumulate any higher than eleven (11) feet below the top of the dam).

As in previous schemes the downstream face of the spillway would require repair with gunite and plugging of the foundation leak.

To provide for the ability to control the lakes level install sluice gates as illustrated in sketch (SK-2) at bays 4 and 5. Bay 4 and 5 are the two bays at the south end of the power house. Repair the leaks between the turbine bays and the generator room. Drill three or four 6-inch diameter holes thru the generator room floor to drain the standing water and prevent future pooling of water.

The estimated cost of Scheme III is:

Installation of Rock Anchors @ 15'-0" c/c	\$ 74,300
2 - 48" Gates with Operators, etc.	\$ 71,000
Repair of Spillway	\$ 93,500
TOTAL	\$238,800

We recommend that you consider Scheme III which is the installation of rock anchors at 15'-0" c/c, installation of two gates and resurfacing of the downstream face of the spillway.

Before this rehabilitation program is started there should be verification of the bedrocks ability to accept the rock anchor forces. We propose that at least three borings be made thru the top of the dam vertically down into bedrock. Also, concrete cores should be taken and tested for strength.

In previous conversations we had mentioned that installation of a gate through the bottom of the dam would be considered. This is not feasible.

If you have any questions about this report, please let us know.

Very truly yours,

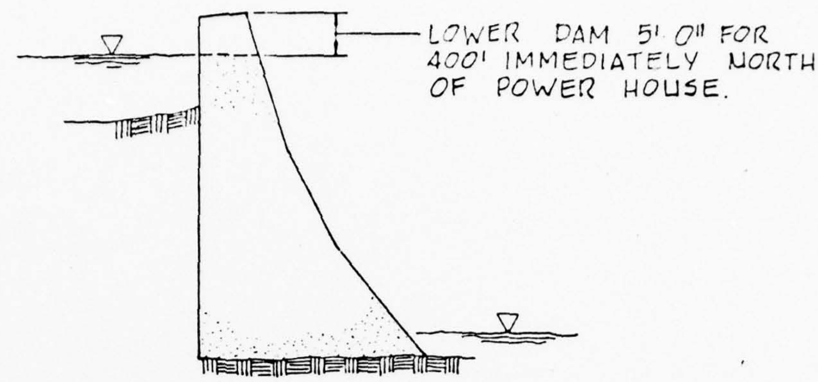
WILEY & WILSON, INC.

W. B. Nolen

W. B. Nolen, PE

WBN/pa
Enclosure (SK-1,SK-2)
cc: R. C. Dodl, Jr., PE

1



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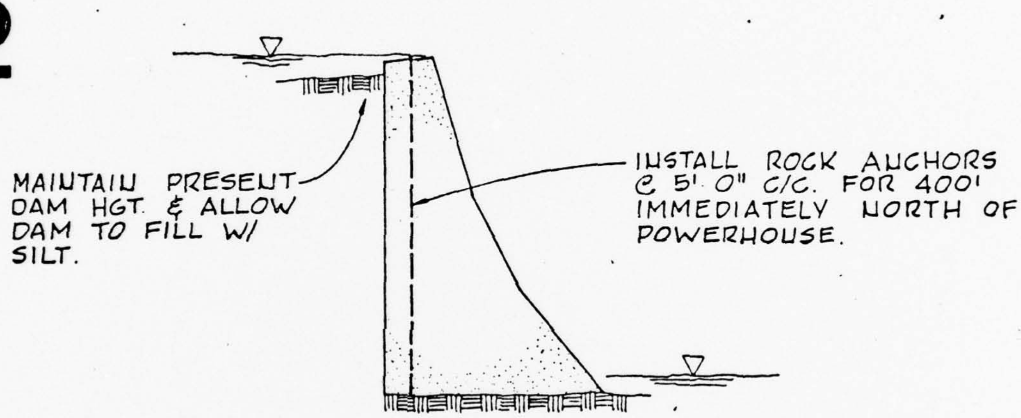

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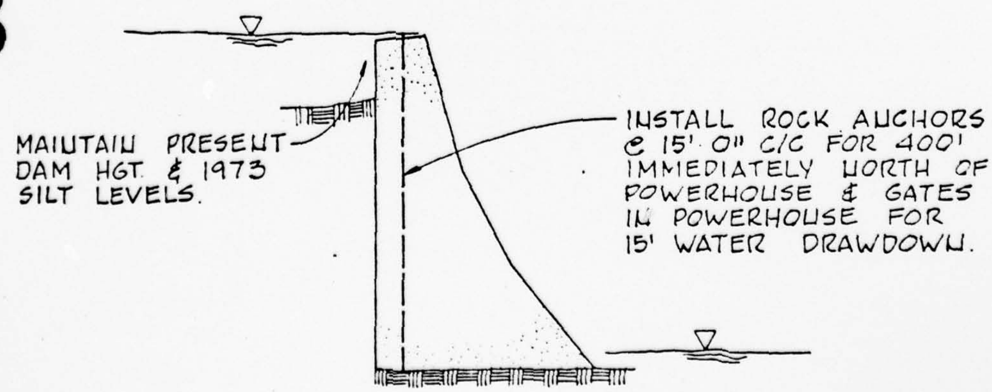
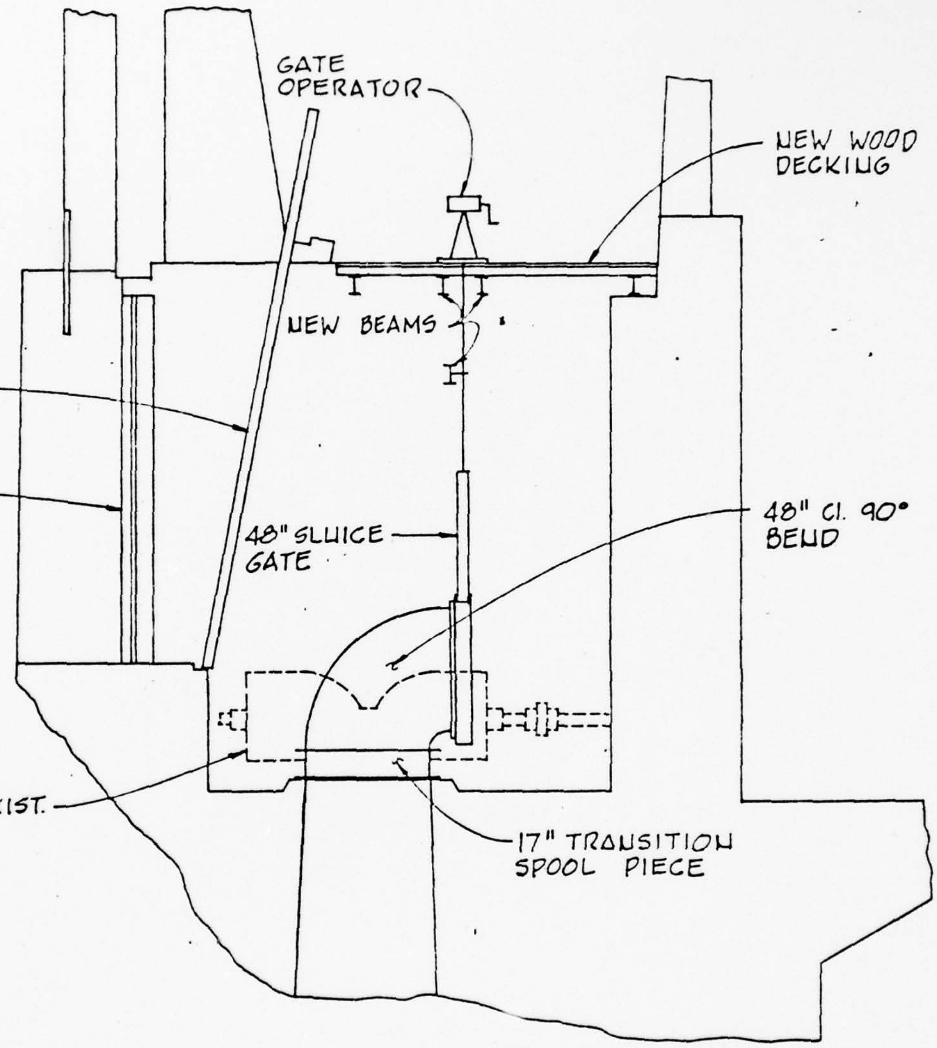


DATE:	9-6-74	DAM CROSS SECTIONS EMPORIA DAM - EMPORIA, VA.	COMM. NO. 3010	
DRAWN BY:	REC		NUMBER:	SK-1
APPROVED BY:	W.B.M.	WILEY & WILSON, INC. LYNCHBURG, VA.	SCALE:	1" = 20'



SECTION · POWER HOUSE

DATE: 9·6·74	GATE INSTALLATIONS @ EXIST. POWER HOUSE EMPORIA DAM, EMPORIA, VA.	COMM. NO. 3010	
DRAWN BY: REC	WILEY & WILSON, INC. LYNCHBURG, VIRGINIA	NUMBER:	SK-2
APPROVED BY: W.B.M.		SCALE:	NONE

AD-A063 638 ARMY ENGINEERING DISTRICT NORFOLK VA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. MEHERRIN RIVER DAM AT EMPORIA (VA --ETC(U)
AUG 78 H E STRAWSNYDER

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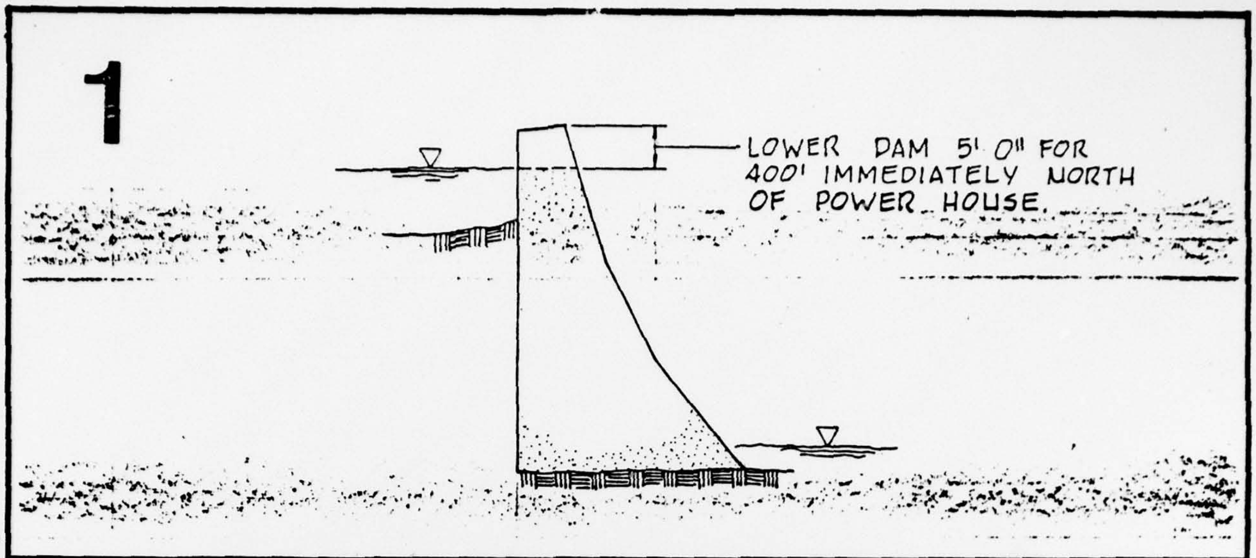
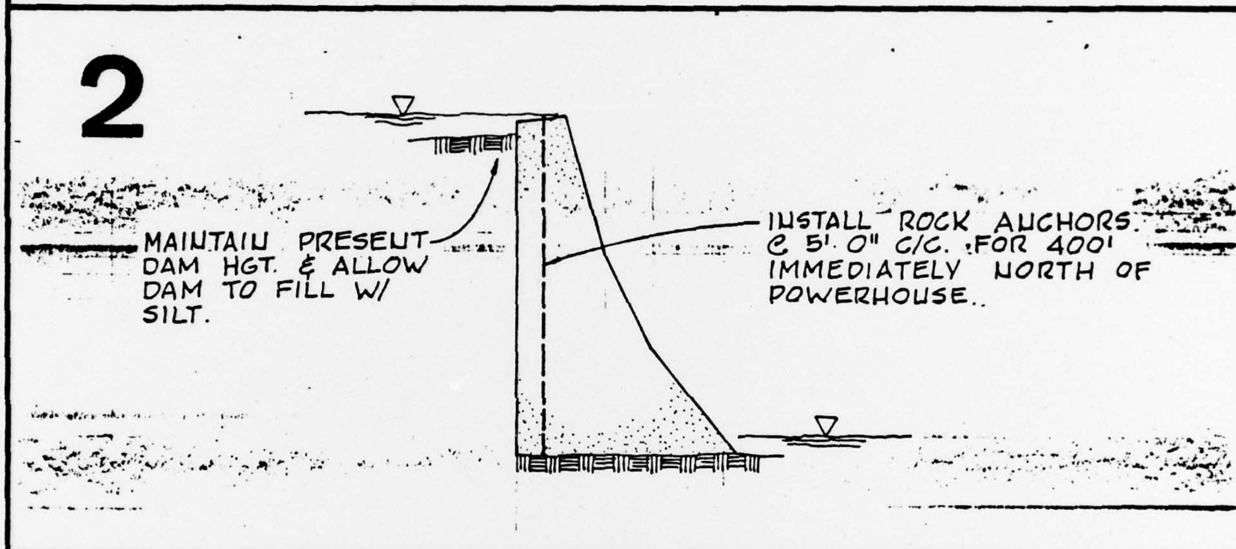
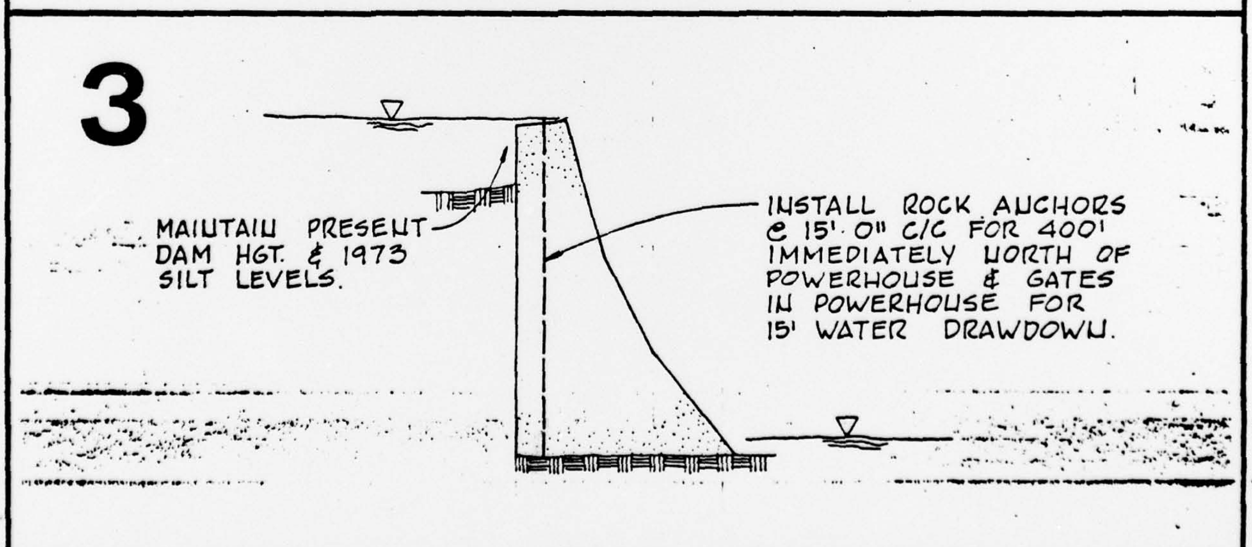
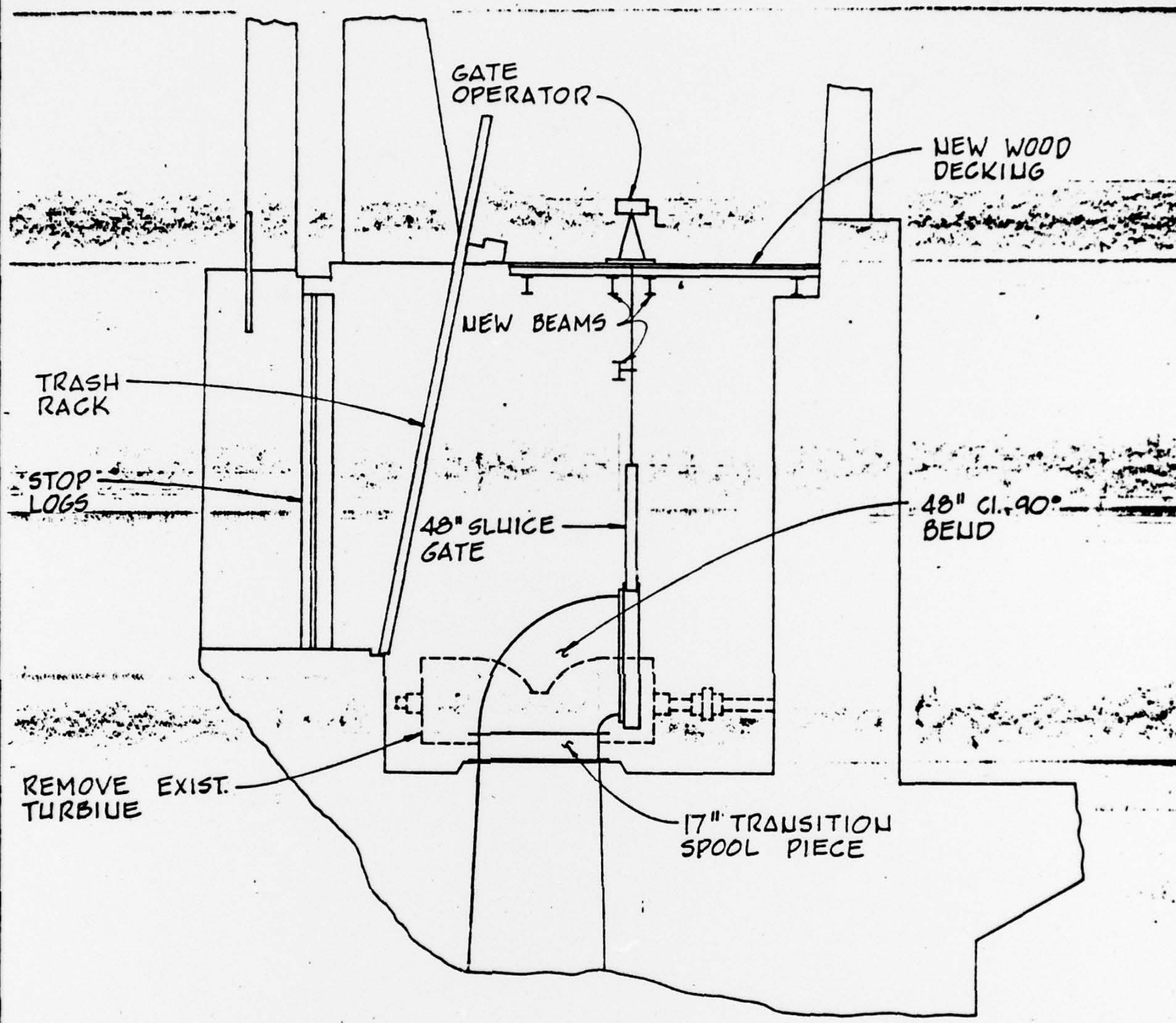






DATE:	9-6-74	DAM CROSS SECTIONS EMPORIA DAM - EMPORIA, VA.	COMM. NO. 3010	
DRAWN BY:	REC	WILEY & WILSON, INC. LYNCHBURG, VA.	NUMBER:	SK-1
APPROVED BY:	W.B.M.		SCALE:	1" = 20'



SECTION · POWER HOUSE

DATE:	9·6·74	GATE INSTALLATIONS @ EXIST. POWER HOUSE EMPORIA DAM, EMPORIA, VA.	COMM. NO. 3010	
DRAWN BY:	REC	WILEY & WILSON, INC. LYNCHBURG, VIRGINIA	NUMBER:	SK-2
APPROVED BY:	W.B.M.		SCALE:	NONE

WILEY & WILSON, INC.
A PROFESSIONAL CORPORATION
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LYNCHBURG — RICHMOND — VIRGINIA BEACH

2310 LANGHORNE ROAD
P. O. BOX 877
LYNCHBURG, VA. 24505
804-847-0102

December 17, 1976

Mr. Robert K. McCord
City Manager
City of Emporia
P. O. Box 511
Emporia, Virginia 23847

Re: City of Emporia, Virginia
Meherrin River Dam
Comm. No. 5189

Dear Mr. McCord:

In accordance with the Agreement for Engineering Services and subsequent correspondence (see letter of December 18, 1975) we are submitting the following report on the above project. The purpose of this report is to update the original engineering study and modify the recommendations for improvements and repairs to the Meherrin River Dam based on the results of the work done by the Contractor under Stages 1 and 2 of Division I of the project.

The work under Division I was divided into three stages in order to permit an investigation of the condition of the dam before proceeding with other repair work. Stage 1 consisted of the drilling and grouting of seven core holes spaced at 60' intervals across the spillway section of the dam. Stage 2 consisted of the installation and testing of seven rock anchors at the same 60' spacing. Stage 3 consists of the drilling

and grouting of twenty-three additional holes and the installation of twenty-three additional rock anchors making the final spacing between rock anchors fifteen feet.

The work under Stage 3 is now underway. Before commencing with each stage, the results of the information obtained in the previous stage was analyzed to determine whether to proceed with the succeeding stage. A geological report dated October 4, 1976 by Geotechnics, Inc., consulting geologists for the project, gives a technical description of the concrete and rock cores obtained during the core drilling in Stage 1 and the results of the compressive tests made on samples of the concrete and rock cores. This report also presents an evaluation of the condition of the dam and the geology of the foundation for the dam. A copy of the geological report has been submitted to the City.

A study of the information obtained in Stage 1 indicated that although there was deterioration in the dam, the conditions of the concrete and bedrock were such that it was advisable to proceed with Stage 2. The Contractor, Cunningham Core Drilling & Grouting Corporation, was authorized to proceed with the installation of the seven test rock anchors included in Stage 2 of the project. All seven rock anchors were installed and tested and were found to meet the requirements of the specifications for tensile loading. The objective of the seven test rock anchors was to determine whether the specified anchors could effectively "take-up" in the rock and tie the concrete dam to the bedrock, thus increasing the factor of safety against overturning.

The testing program conducted in Stages 1 and 2 verified that the concrete in the dam and the bedrock were adequate in strength and that the dam could be repaired.

Based on these results the Contractor was authorized to proceed with Stage 3 and a number of the additional twenty-three core drilled holes have now been completed providing additional information on the condition of the dam. This core drilling of approximately thirty holes spaced on 15' centers has identified one important deficiency of the dam. The contact zone between the concrete structure and the bedrock is weathered and deteriorated, especially in the area between the fish ladder and north abutement. Both the bedrock and the concrete are deteriorated in the contact zone but the concrete is in worse condition. The concrete along the top or the cap of the dam has experienced some weathering varying from eight inches to one foot in depth. There are several other smaller areas where some deterioration in the concrete has occurred. Except for the contact zone between the concrete and rock, the foundation rock has been found to be in good condition and suitable for the installation of the rock anchors.

Logs of all of the cores plus a drawing of the downstream elevation of the dam showing the areas where additional grouting is needed are being prepared by the geologists and will be submitted to the City for the City's records.

Based on the information now known about the dam from the core drilling and grouting which has been completed we are modifying our recommendations for the remaining repair work. In our report of September 10, 1974 we recommended complete resurfacing of the downstream face of the spillway section of the dam. In the development of plans and specifications for the project we had planned to include the resurfacing in Division II of the project. From the information we now have on the conditions within the dam, we believe it more important to make additional internal repairs and reduce the amount of surface repairs.

Our present recommendations for additional repairs to the dam are as follows:

(1) Additional core drilling down into sound bedrock and grouting of the contact zone between the concrete structure and the bedrock. This will entail approximately twenty additional core holes in the north end of the spillway spaced between the existing rock anchors, making the core and grouting holes 7.5 feet apart in this area of the dam. The additional grouting in this section will help prevent further rapid deterioration of the concrete and bedrock due to the various processes of weathering.

(2) Seal the large leak under the dam just north of the fish ladder by additional massive grouting.

(3) Drill approximately two core holes into sound bedrock at the north abutement and grout as required. The existing borings near this end of the dam indicate that the bedrock was of poor quality when the original dam was constructed. Grouting will help close off voids in the concrete and bedrock.

(4) Drill and grout approximately three holes in the south end of the dam between the powerhouse and the south abutement. Original plans did not include any grouting at the south end of the dam, but in view of the conditions encountered at the contact zone between the dam and bedrock in other sections of the dam, we now feel that some drilling to verify the conditions of the south end of the dam and some grouting of the contact zone should be done.

These repairs involve the same type of work now being done by the contractor for Division I and we believe that the most feasible and economical method of accomplishing these repairs would be to add this work to the present contract by a change order. Therefore, if the City concurs in these recommendations for additional repairs, we are requesting authorization from the City and the Farmers Home Administration to obtain a quotation from Cunningham Core Drilling & Grouting Corporation to do the additional work as outlined above. It will be necessary to do the work on a unit price basis, since exact quantities cannot be determined in advance.

Our modified recommendations for Division II of the dam repair project include the following:

(1) Make repairs to the spillway surface where large cracks or voids exist. The purpose of these repairs would be to restore the areas of the spillway which have been weakened structurally due to cracking and deep spalling (areas deteriorated twelve inches or deeper). The work would consist of removing the deteriorated concrete from the cracked or spalled areas to sound concrete, applying an epoxy bonding agent and then filling the voids with new concrete. The procedure would require forming small areas scattered over the face of the spillway for the placing of concrete.

(2) Divert the surface water away from the north abutement of the dam by excavating an appropriate berm ditch. Considerable surface water from the adjacent area now flows into the river along the downstream face of the north abutement of the dam and is causing unnecessary erosion along this abutement.

Division II of the project would be accomplished under a separate contract based on competitive bids received in accordance with Farmers Home Administration requirements. We are hereby requesting the City's authorization to proceed with the plans and specifications for Division II of the project so as to permit bidding of this phase of the work in late winter or early spring 1977.

Our modified recommendations for Division II of the dam repair project include the following:

(1) Make repairs to the spillway surface where large cracks or voids exist. The purpose of these repairs would be to restore the areas of the spillway which have been weakened structurally due to cracking and deep spalling (areas deteriorated twelve inches or deeper). The work would consist of removing the deteriorated concrete from the cracked or spalled areas to sound concrete, applying an epoxy bonding agent and then filling the voids with new concrete. The procedure would require forming small areas scattered over the face of the spillway for the placing of concrete.

(2) Divert the surface water away from the north abutement of the dam by excavating an appropriate berm ditch. Considerable surface water from the adjacent area now flows into the river along the downstream face of the north abutement of the dam and is causing unnecessary erosion along this abutement.

Division II of the project would be accomplished under a separate contract based on competitive bids received in accordance with Farmers Home Administration requirements. We are hereby requesting the City's authorization to proceed with the plans and specifications for Division II of the project so as to permit bidding of this phase of the work in late winter or early spring 1977.

It is understood that the work under Division III for the installation of outlet control valves will be deferred until additional funds are available.

It is difficult to accurately estimate the cost of the additional core drilling and grouting because of the unknown quantities involved; however, a rough estimate indicates that the additional cost (less the underrun on Stages 1 through 3) will amount to approximately $40,000. This would make the total cost of Division I, including the original Cunningham Core Drilling & Grouting Corporation contract, engineering costs, geological costs and a small allowance for contingencies, approximately $273,000. Subtracting this amount from the original $375,000 sum available, leaves $102,000 for the repairs to the surface of the spillway under Division II.

We are enclosing twelve copies of this report plus one additional copy of the geoglogical evaluation by Geotechnics, Inc. If you find the report satisfactory please forward one copy of our report plus the copy of the geological report to the Farmers Home Administration for their review and concurrence.

Very truly yours,

WILEY & WILSON, INC.

R. C. Dodl, Jr.

R. C. Dodl, Jr., PE

RCD:vs
Enclosures